VOL. 56. APRIL, 1930. No 4.

PAPERS, REPORTS, DISCUSSIONS, AND MEMOIRS

This Society is not responsible for any statement made or opinion expressed in its publications.

CONTENTS

Papers: PAGE

Completion of Moffat Tunnel of Colorado.
By CLIFFORD ALLEN BETTS, M. AM. SOC. C. E. 679

Frequency and Intensity of Excessive Rainfalls at Boston, Massachusetts.
By CHARLES W. SHERMAN, M. AM. SOC. C. E. 717

The Shannon Power Development in the Irish Free State.
By A. R. C. MARKL, JUN. AM. SOC. C. E. 727

Reports of Committees:

Ice as Affecting Power Plants: Final Report of Committee of Power Division 743

Discussions:

Report of the Committee of the Irrigation Division on A National Reclamation Policy.
By MESSRS. F. N. CRONHOLM, R. I. MEEKER, and E. B. DEBLER 761

Pumped-Storage Hydro-Electric Plants.
By WILLIAM W. K. FREEMAN, ASSOC. M. AM. SOC. C. E. 775

Problems Concerning Elastic Stability in Structures.
By S. TIMOSHENKO, ESQ. 779

The Surveying and Mapping Activities of the Federal Government.
By T. W. NORCROSS, ASSOC. M. AM. SOC. C. E. 783

Good Maps Are a Sound Investment.
By L. R. LOHR, M. AM. SOC. C. E. 785

Water Supply from Rainfall on Valley Floors.
By CHARLES H. LEE, M. AM. SOC. C. E. 787

Silt Transportation by Sacramento and Colorado Rivers, and by the Imperial Canal.
By IVAN E. HOUK, M. AM. SOC. C. E. 801

Analytical Solution of Masonry Domes: Unbalanced Loads.
By MESSRS. A. FLORIS and DAVID C. COYLE 809

Regulation of Levels, Flow, and Navigation, Niagara River: Summary and Conclusions of Various Studies.
By F. G. RAY, M. AM. SOC. C. E. 817

Progress Report of Committee of the Sanitary Engineering Division on Friction of Sewage Sludge in Pipes.
By WILLIAM CLIFFORD, ESQ. 821

Report of the Committee of the Board of Direction on Essential Facts Concerning the Failure of the St. Francis Dam.
By MESSRS. B. F. JAKOBSEN, M. H. GERRY, JR., F. B. MARSH, and F. C. FINKLE 827

Adjustment of Transit and Stadia Traverses.
By MESSRS. PAUL H. UNDERWOOD, F. H. PETERS, and H. PARRY 847

Factors Governing the Location of Airports.
By MESSRS. N. H. ANGELL, J. OGDEN MARSH, ARCHER R. NORCROSS, JOHN NOLEN, S. N. GRIMM, R. U. ST. JOHN, and JOHN F. PARTRIDGE 855

High Dams: A Symposium.
By MESSRS. B. F. JAKOBSEN, ALLEN HAZEN, and H. DE B. PARSONS 867

Construction of the James River Bridge Project.
By WILSON T. BALLARD, M. AM. SOC. C. E. 875

General Specifications for Steel Railway Bridges: Prepared by the Conference Committees from the American Society of Civil Engineers and the American Railway Engineering Association.
By MESSRS. C. H. CHAPIN, HUNTER MCDONALD, A. F. BLAESS, and C. R. HARDING 881

Plastic Flow in Concrete Arches.
By MESSRS. B. F. JAKOBSEN and C. A. P. TURNER 887

Relation Between Rail and Waterway Transportation: A Symposium.
By LEWIS C. SORRELL, ESQ. 895

Memoirs:

EDWARD AUSTIN BOND, M. AM. SOC. C. E. 903
FREDERICK AUSBERT DELAVAU, M. AM. SOC. C. E. 904
DANIEL WINGERD GROSS, M. AM. SOC. C. E. 905
HENRY SCUDDER JAUDON, M. AM. SOC. C. E. 906
JOHN WILLIAM LIEB, M. AM. SOC. C. E. 907
WILLIAM CARY HATTAN, ASSOC. M. AM. SOC. C. E. 913
EMIT COIN NICKEY, ASSOC. M. AM. SOC. C. E. 915

For Index to all Papers, the discussion of which is current in *Proceedings*, see the second page of the cover.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

COMPLETION OF MOFFAT TUNNEL OF COLORADO*

By Clifford Allen Betts,† M. Am. Soc. C. E.

Synopsis

The 6.2-mile Moffat Tunnel through the Rocky Mountain Continental Divide, 50 miles west of Denver, Colo., was officially opened to railroad traffic on February 26, 1928.

The story of its conception many years ago and of the struggles since 1902 leading to its ultimate accomplishment has attracted widespread attention and has given the undertaking an unusually dramatic background. Difficulties encountered during construction due to caving ground and inflows of water led to the development of new tunneling methods which, combined with the unique Improvement District method of financing, added to the interest. Furthermore, the fact that this combination railroad and water tunnel embodied, at the time of its construction, the longest railroad tunnel in the Americas, in addition to a conduit of equal length, permitted the use of new and modern machinery and led to numerous refinements in standard equipment.

The history of the early stages of the work up to August, 1925, together with a discussion of the contract, has been published by the Society.‡ Inasmuch as that paper made no reference to the survey methods used in carrying alignment and grades over the range that rises to 12 000 ft. above sea level and 2 800 ft. above the tunnel, nor to the camps, these are described herein in some detail, together with other phases of the work which have developed subsequent to 1925.

Soft ground encountered for a distance of 2½ miles in from the West Portal was the outstanding feature of the construction. The methods and machinery used to combat this obstacle, including the design of heavy tim-

Note.—Written discussion on this paper will be closed in **August, 1930,** *Proceedings.*

* For a general description of the history, development, and progress of this undertaking to the summer of 1925, reference should be made to the paper by R. H. Keays, M. Am. Soc. C. E., entitled "Construction Methods on the Moffat Tunnel," *Transactions,* Am. Soc. C. E., Vol. 92 (1928), p. 63.

† Engr., U. S. Bureau of Reclamation, Owyhee Dam, Owyhee, Ore.

‡ "Construction Methods on the Moffat Tunnel," by R. H. Keays, M. Am. Soc. C. E., *Transactions,* Am. Soc. C. E., Vol. 92 (1928), p. 63.

bering, steel reinforcement, concrete lining, and the use of the "Lewis Traveling Cantilever Girder" are among the tunneling developments described. Similarly, the notes on plant machinery and on costs of vital operations epitomize extensive studies.

Ventilation tests made at the Moffat Tunnel offered a unique opportunity for the correlation of data on air-friction factors for tunnels of mining size on which exhaustive information is available and on factors for standard railroad tunnels on which fewer tests have been made.

The construction of the Moffat Tunnel involved among other quantities (in round figures): 750 000 cu. yd. of excavation; 16 000 000 ft., board measure, of timber; 2 500 000 lb. of dynamite; 700 000 lb. of drill steel; 700 miles of drill holes; 28 000 000 kw-hr. of electrical energy; and 25 000 000 man-hours.

The final cost of work as shown by outstanding bond issues is $15 470 000, in addition to which approximately $2 000 000 revenue from camps, commissaries, interest on bank deposits, and other sources went into the work and into interest on the bonds during construction.

The ultimate object of the project is to make possible a transcontinental rail route through Denver; to insure adequate transportation facilities for Northwestern Colorado; and to provide an additional source of water supply for Denver and contiguous territory.

Historical Résumé

Early Surveys.—When the final "holing through" blast, touched off by President Coolidge on February 18, 1927, broke down the 7-ft. granite barrier between the east and west headings of the Moffat Water Tunnel, it was said, as the Colorado and Utah officials joined hands, that this was the highest and lowest holing through of a tunnel in history; the highest because it was more than 9 200 ft. above sea level, and the lowest because the headings were 2 800 ft. below the crest of the Continental Divide.

For half a century this Divide, extending from the north to the south boundaries of Colorado, and beyond, had stood in the way of adequate transportation from the East to the West. A tunnel through the Rockies to provide a practical route had been first conceived in the early Sixties by Governor Gilpin when the Union Pacific Railroad Company began laying the first transcontinental track. It had been taken for granted that the route would pass through Denver, the largest center of the West. The late Grenville M. Dodge, Hon. M. Am. Soc. C. E., Chief Engineer of the Union Pacific Railroad, had surveyed all the passes from Laramie Canyon in Wyoming to the Arkansas River in Southern Colorado; but the unbroken barrier of the Divide rising to 12 000 ft. above sea level offered no suitable pass. On November 7, 1866, General Dodge's party barely escaped death in a blizzard near the site of the present East Portal of the Moffat Tunnel by turning their pack animals loose and going down the valley to the nearest camp on foot. This unfortunate experience did not show this site in a favorable light at the time, but eventually the process of elimination proved it to be the narrowest part of the Divide at an elevation suitable for railroading.

Surveys were run over this Rollins Pass again in 1867 for the Jefferson and Boulder County Railroad and Wagon Road and in 1869 by the Kansas Pacific Railroad Company. In 1880, a 2-mile tunnel site was surveyed and an open cut begun near Yankee Doodle Lake. This and some grading constitute the remaining evidences of the ambitious program of the Denver, Utah, and Pacific Railroad Company which, however, abandoned the work. Part of the grade was later acquired by the Chicago, Burlington, and Quincy Railroad Company, and in 1884-85 that Company ran surveys over Rollins Pass. A 3-mile and a 5-mile tunnel were projected; the east end of the latter being above and to the north of East Portal and the west end on Jim Creek west of James Peak.

Moffat Road Surveys.—Following this the Moffat Road ran ten lines in 1902, locating a 2.6-mile, a 4.1-mile, and a 5.8-mile tunnel. Plans went so far that bids were obtained on the 4.1-mile tunnel but, believing that the time required for construction would be prohibitive and that only the long tunnel would eliminate effectively the snow hazards, the road was built over Rollins Pass at Elevation 11 660, reaching Corona on October 2, 1904. It was hoped that, pending construction of a tunnel, the scenic value of the line over the Rockies would compensate for the snow-fighting costs; but the advent of the automobile interfered, and the bitter fight against Nature forced the road into receivership by the time the Moffat Tunnel was begun in 1923.

David H. Moffat's historic fight against ruthless Nature and adverse railroad interests is an epic of vision, determination, and public-spirited generosity. His road reached Steamboat Springs on December 13, 1908, having cost $14 000 000 and later was extended to Craig, Colo. (255 miles from Denver).

Construction

Moffat Tunnel Surveys.—The Moffat Tunnel Act authorizing the construction of the Moffat Tunnel provided no funds that could be used for preliminary investigations, such as diamond drilling and surveys, prior to the sale of bonds. Enough private funds were made available for preliminary surveys, however, and thus the old Denver and Salt Lake Railroad (Moffat Road) survey lines were re-established and monumented. Upon geological advice that it would be advantageous to shift the West Portal location about 400 ft. south and the East Portal 75 ft. north, new portal sites were located by offsets from the old line. Timber cuttings for the new line were then started while the tunnel was being extended into the mountain on the offset lines.

The surveys for the Moffat Tunnel crossed the Continental Divide at a storm-swept point 12 000 ft. above sea level. There the crest of the Rockies flattens out, forming a plateau ½ mile wide which effectively prevented sighting toward each portal from a point on top. Consequently, it was necessary to establish three points on the Divide so located that from the easternmost point the East Portal back-sight target could be seen (Fig. 1), while from the western point the West Portal terminal target was visible. These five points, forming the center line of the Water Tunnel, had to be placed in alignment at night because there was considerable refraction during daylight hours between the targets at elevations of 9 000 and 12 000 ft. This was attributed

largely to the differing densities of the air and to temperature variations. Frequently, it exceeded 3 ft. on the 6-mile sight between the east terminal target and the Divide.

Lights placed behind paper or ground glass gave excellent results as sights especially after midnight when atmospheric conditions were most favorable. To permit accurate setting of points at night, a spirit-level head was mounted on a standard transit tripod with a painted metal target mounted vertically at one end so that a light could be placed behind and the target moved until it was on line. A hole in line with the pointed aperture of the target carried a plumb-bob string for setting points (using wind breaks). The final underground closure of 0.11 ft. after carrying the line 10 miles over the range and 6 miles underground substantiates the accuracy of this night work.

The line was carried underground through the Water Tunnel (which served as a pioneer bore), by setting up the transit on the terminal target, sighting on the skyline target on top of the Divide, depressing the telescope, and setting a point at the portal. From there the line could be carried in on special brass vernier bars, ¼ in. by 1 in. by 10 in., attached to plugs in the roof of the tunnel. These bars were accurately graduated in hundredths of a foot so that a brass vernier rider carrying a plumb-bob line could slide along the upper edge and permit readings to 0.0001 ft. These bars at about 500-ft intervals not only made it easy to run independent lines and subsequently to compare the readings, but also facilitated adjustments where the ground and timbers were in motion. Caps moved several inches at points between 2 and 3 miles in from West Portal.

To determine the distance between portals a triangulation system was used, with a base line 3 756.587 ft. long, near West Portal, and eight stations on the mountains. Two U. S. Geological Survey, 300-ft. invar tapes having a coefficient of expansion of only 0.000 000 42 were used in measuring the base line, with corrections for sag and temperature. Supplementing the triangulation, slope measurements were made over the mountain with a 500-ft. chain, thus locating the profile. The horizontal distance through the tunnel was 1.75 ft. shorter than that computed.

Precise levels were run over the center line between the portals. These checked previous grades run independently over the Divide *via* the survey clearing and the Denver and Salt Lake Railroad tracks (31 miles). The grades checked within 0.345 ft. on holing through and were readily adjusted in the remaining enlargement (5 523 ft.).

Cross-sections of the Railroad Tunnel were taken with a "sunflower", mounted on a tripod, which consisted of a vertical aluminum disk graduated in degrees, beginning with zero at the top. Pivoted at the center was a rotating arm upon which a measuring rod could be placed to show simultaneously the distance and angle to a point on the tunnel wall. From the known positions of the "sunflower", sections could be plotted rapidly.

Work Begun by Commission.—Ground was broken for the Water Tunnel on August 25, 1923, following the decision of the U. S. Supreme Court validating the Moffat Tunnel Improvement District Law of May 12, 1922, whereby the Improvement District, of 9 000 sq. miles, including Denver, was created.

Through its elected officers, the Moffat Tunnel Commission was empowered to assess real estate in the District, issue bonds, construct a railroad and water tunnel, and administer them when finished. This Colorado law provided for the making of contracts, not exceeding 99 years in duration, for the use of the tunnel up to the limits of its capacity for rail and water transportation as well as for power, telegraph, and telephone lines, at the same time prohibiting any monopoly or the sale of the property.

Pending the letting of a contract and in order to have the pioneer tunnel underground before the deep snows indigenous to the 9 000-ft. elevation arrived in October, the Moffat Tunnel Commission rushed the construction of camps at both East and West Portals. When a contract was awarded to Hitchcock and Tinkler, Incorporated, on September 20, the camps were under roof, the West Portal Water Tunnel had been driven in 32 ft., and the open cuts were well advanced.

West Portal Camp.—Both East Portal and West Portal camps were constructed in 1923 while machinery was being delivered and the excavation of the tunnel started. The camp sites were so different in appearance that it was frequently said that the portals hardly seemed to be in the same mountain. This difference persisted as the headings extended underground. West Portal, the first camp completed, was surrounded by heavily timbered slopes beneath which lay the soft rock of an extensive fault zone, while East Portal had more barren and precipitous surroundings (Fig. 1), showing the solid gray granite which extended to the center of the Divide.

While the camp site at West Portal was being cleared of heavy pine and the buildings were being constructed, the men were quartered at Wood's saw-mill near-by. The water supply for this site was taken from Buck Creek and piped by gravity to camp and plant buildings, provision being made for fire hydrants in the camp streets. Large septic tanks were used for the sewerage system. The ten 30-man bunk-houses, 20 by 50 ft., with rooms accommodating two men each and allowing 250 cu. ft. of air per man, were built 75 ft. apart to reduce the fire hazard.

These houses were wired for electric lights, heated by pipeless hot-air furnaces, and built with paper between the inside and outside sheetings. A building was provided for the Moffat Tunnel Commission Engineering Staff and one for the Contractor's organization. Later, these were augmented by eight 4-room cottages for engineers and foremen with families and by twenty-two 2-room cabins for miners with families, preference being given by seniority. The cottages rented for $25 per month and the cabins for $15, which practically amortized the cost of construction. Many families lived in privately owned houses near the portal camps.

A mess hall seating 250 and serving as many as 40 000 meals per month, at a cost of about $0.42 each, was constructed between the bunk-houses and the portal, as was also a recreation hall containing a commissary, post office, reading room, pool hall, and auditorium which could be used for dances, prize fights, moving pictures, and other entertainments for which the camp gained a reputation. In fact, East Portal and West Portal, with about 500 workmen at

each, ranked with some of the largest communities along the railroad in money spent during the five years of bustling activity before construction ceased and they became "ghost cities".

The laundry, equipped with rows of stationary tubs, shower baths, toilets, and plenty of hot water, was the central wash house, there being no tubs in camp. As a preventive of pneumonia, which at that altitude is pernicious, a change house was built at each portal with lockers and showers so that workmen coming off shift could enter the building and change to dry clothes before going out into the cold. Wet clothes were attached to wire rope hoists and raised out of reach overhead to dry, the free end of each rope being secured inside the owner's locker.

Grade schools accommodating 60 pupils were provided for the camp children at each portal. The Moffat Tunnel Commission furnished the buildings, and the County School Districts, the teachers.

East Portal Camp.—The camp at East Portal (Fig. 1) was planned similar to the West Portal camp, except that the rocky ground led to the installation of individual septic tanks for the houses instead of ditching for sewers to a central disposal plant.

Plant Buildings.—The plant buildings at each portal were grouped closely around the portals and connected to the Water Tunnel by snowsheds so that the severe winter weather, that frequently extends into June at this locality, would not handicap operations.

A steel sharpening shop opened directly off the tunnel. Beyond this was the machine shop with lathe, drill press, grinders, welding outfits, and all tools required for large or small repair work.

The power house received 2 300-volt current transformed from the 44 000-volt, 3-phase, 60-cycle transmission line from the Boulder Canyon Plant of the Public Service Company of Colorado. Power consumption was planned to keep the maximum demand as low as possible, usually about 1 800 h.p., with the result that the cost averaged 1 cent per kw-hr. for a total use of about 28 000 000 kw-hr.

Comparatively few power interruptions occurred on the power line to East Portal, but the 7-mile line between portals, over the Continental Divide, required considerable maintenance during winter months. Tubes of ice formed on the wires by snowdrifts and sleet storms and then, swayed by the high gales, broke off insulators or poles. This usually occurred at the highest part of the line where iron wire replaced copper for strength. On one occasion a Mexican laborer tripped across a section of the power line that was buried in snowdrifts and was burned beyond recognition.

Adjacent to the power house was the electricians' shop and the battery recharging station for the storage batteries used on the 4-ton locomotives. The warehouse, primer house, and carpenter shop were grouped near-by.

At West Portal a timber-framing shed was equipped not only with the usual cut-off and band-saws but also with a timber dapping machine consisting of a dado-head saw that could be drawn across the timbers, cutting to desired depths and widths.

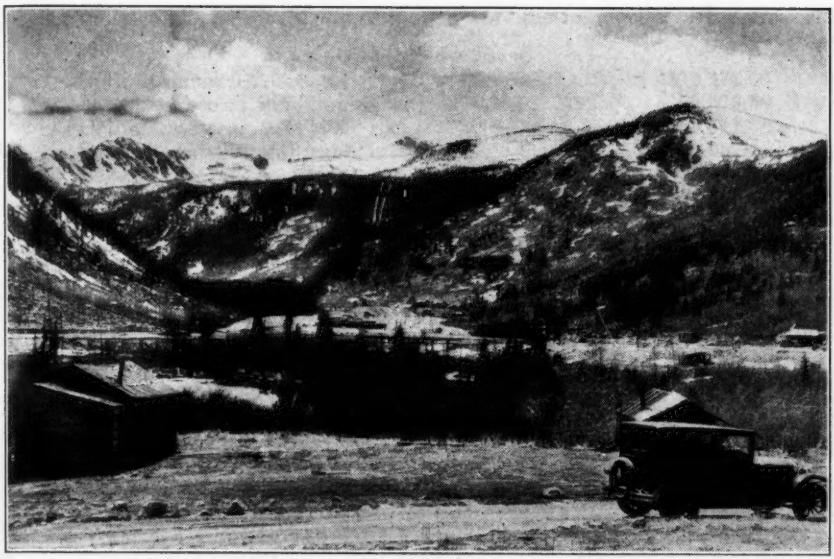


FIG. 1.—EAST PORTAL, MOFFAT TUNNEL, LOOKING WEST FROM BOULDER CREEK. CAMP BUILDINGS IN CENTER AT FOOT OF MOUNTAIN. PARALLEL MARKS OF SURVEY LINE APPEAR STRAIGHT AHEAD FROM WOODED SLOPE.

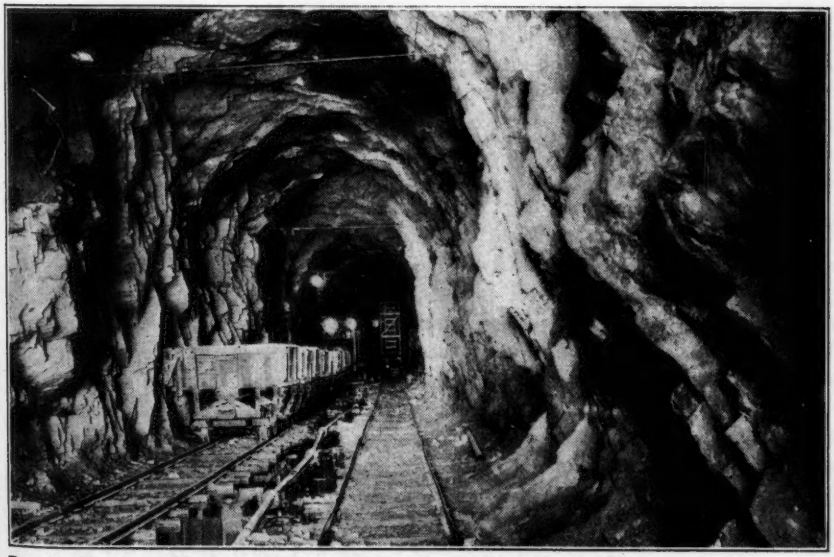


FIG. 2.—TYPICAL RAILROAD TUNNEL THROUGH GRANITE, SHOWING EQUIPMENT. LATER, 36-INCH GAUGE TRACKS WERE REPLACED BY BALLASTED STANDARD SINGLE-TRACK LINE.

Among the items included in the plant list used in advertising the machinery for sale at the conclusion of the job were the following, with their costs, f. o. b. job:

Rolling stock	$213 171.00
Air shovels and mucking machines	62 573.50
Drills	53 541.08
Concrete mixers and guns	25 653.83
Lewis girders	26 782.00
Motor generator sets	20 848.96
Pumps	8 612.00
Blowers	14 279.00
Equipment in shops and power houses	103 000.00
Miscellaneous electrical equipment	12 000.00
Hoists and jacks	11 795.67
Rail	30 180.00
Wire	54 322.00
Pipe	58 134.89

The "Inventory of Plant Machinery, Camp Buildings, and Equipment" showed a summary of:

Total plant machinery	$739 499.01
Total camp buildings (East Portal)	100 096.48
Total camp buildings (West Portal)	124 926.18
Total camp equipment (East Portal)	13 200.00
Total camp equipment (West Portal)	15 800.00
	$993 521.67

Geological Conditions Encountered.—From the beginning the ground at West Portal presented difficulties. The East Portal headings, on the other hand, passed through comparatively uniform grey metamorphic granite (Fig. 2) with some quartz monzonite and injection gneiss seams of consistently similar dip and strike. In all there was less than 1 500 ft. of seamy ground requiring support east of the apex and at no place were the pressures great. This was the character of formation on which the plans were predicated.

It is interesting to note that this end of the tunnel was completed six months ahead of schedule and practically within the original estimate. The semi-annual financial statement of the Moffat Tunnel Commission of January, 1925, before costs had been greatly increased by the unforeseen difficulties, showed the cost of excavation in the headings to be $10.41 as compared with $17.00, the contract price; and the cost of enlargement excavation, $4.80 as compared with $4.70 bid. (See Appendixes I and II.)

The fact that the east headings overcame the half-mile lead that the West Portal had acquired by starting first and eventually made a mile greater progress indicates the difference in rock at the two ends. Furthermore, nearly twice as much dynamite per yard (11 lb.) was required for the East Portal rock as for that at West Portal.

Geologists had predicted less than 2 000 ft. of questionable rock that might require support; no funds had been allowed for preliminary diamond drilling. As a result the 2½ in. of fault zone with seams and dikes not only increased

the cost of the project, but required more time, as is graphically shown on the progress chart, (Fig. 3). Subsequent geological studies made from the bore within the mountain indicated that the hidden fault zone, which caused the shattered rock at West Portal, extended for such a long distance along the Divide that to have avoided it by shifting the tunnel location would have been impractical.

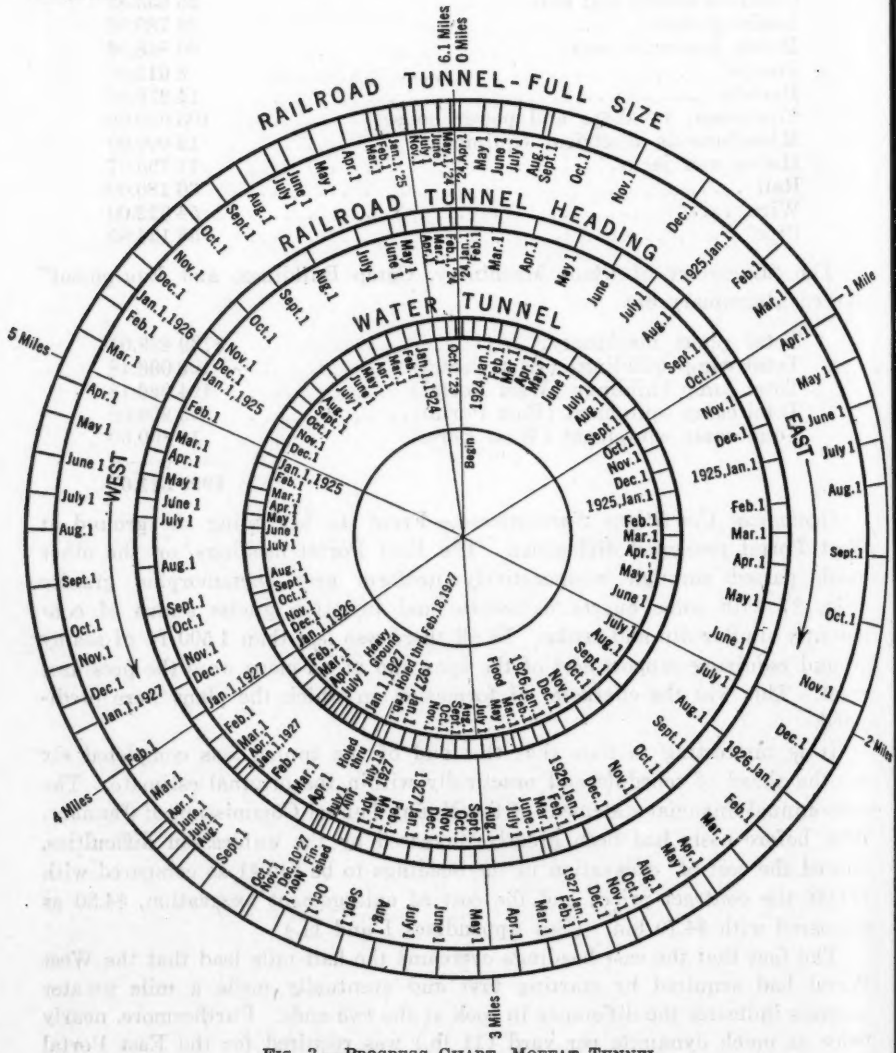


FIG. 3.—PROGRESS CHART, MOFFAT TUNNEL.

Operations at East Portal

Twin Heading System.—At East Portal a system of alternating was established whereby the same crew could drill first the Water-Tunnel heading and

then, while that was being mucked out after shooting, could move over to the Railroad-Tunnel heading and proceed to drill there without interfering with the mucking crew. In this way three shifts of about seventeen men each could usually drill, shoot, and muck three 8-ft. rounds in 24 hours, or a full round each shift. Under this system the maximum advance made during a 30-day period was 1 583 ft. The maximum progress in a single heading with three shifts of one crew each was 849 ft. and the maximum daily advance, 33 ft.

Headings and Cross-Cuts.—The general procedure at the east end followed the pre-arranged plan of driving railroad center headings each way from cross-cuts from the Water or Service Tunnel and, later, enlarging around this center heading to full 16 by 24 ft. size, standard American Railway Engineering Association dimensions. The Water Tunnel, cross-cuts, and main headings were 8 by 8 ft., or the equivalent of 7 ft. high by 9 ft. wide. The cross-cuts were spaced as nearly 1 500 ft. apart as the favorable formations in the rock would permit and, to facilitate transportation, were usually run at an angle of about 60° with the center line of the headings. The cross-cuts are numbered from both ends beginning at the portals; that is, Cross-Cut 8-W indicates the eighth one in from the West Portal. From the Water Tunnel center line to the parallel main-heading center line, 75 ft. to the north, the alignment survey traverses were closed through these cross-cuts. To permit access, headings were kept at least one cross-cut ahead of the enlargement.

Drilling.—The 26 holes, 9 ft. deep, constituting the average heading round could be drilled in 2½ hours. The drilling speed ranged from 10 to 25 in. per min., 1 cu. ft. of air being used per in. The four changes of steel generally used were 2⅜-in. bits for 3 ft.; 2¼-in. bits for 5 ft.; 2⅛-in. bits for 7 ft.; and 2-in. bits for 9 ft.; 1¾-in. Swedish steel gave good results when properly heat-treated.

Mucking by Machine.—Electric mucking machines loaded the 2-yd. cars at the rate of 24 yd. in 2 hours. Inasmuch as 60% of the time required to load cars is consumed in switching, this item becomes very important. Accordingly, diverse schemes were tried to cut down losses, including those due to derailments at the portable California-type jump-switches which were originally in use in connection with a single main-line track. As a result a compact air hoist (Fig. 5) was suspended by a two-wheel trolley from a 3-in. steel jack bar which, in turn, was hung on clevis hooks attached to drill steel in the walls of the heading near the roof. A grapple-hook attached to the bottom of the air hoist could be placed under each end of a 2-yd. car and lift it bodily off the track, so that it could be swung to one side while a loaded car was hauled back from the electric mucker by the electric locomotive. This "empty" was then placed on the track ahead of the locomotive and train and run up to the shovel. Meanwhile, another "empty" was lifted off the track and set over out of line ready for a repetition of the operation, until the entire train was loaded. This switching was done about 200 ft. back from the face. Locomotive attendance, essential to this speedy switching system, was required by the grades anyway, so in this case it was not chargeable to this system of switching.

Hauling.—The high sided 2-yd. cars used on the 24-in. track (40-lb. rail) in the headings were suitable for machine loading and not adapted to hand mucking. In order to relieve these cars of the long trip out on the dump and to release them for the many heading operations which sometimes numbered 20 or more and made heavy demands upon transportation, transfer docks were provided where muck from the narrow gauge cars could be dumped into the 4-yd., 36-in. gauge cars on the railroad tunnel level, 8 ft. below the Water Tunnel grade. Sidings for "empties" were installed at every other cross-cut so that dispatching of transportation might be more flexible. Hauling costs increased perceptibly with the distance in.

Underground Accommodations.—A "Harvey House" (lunch room) in a niche in the rock more than a mile underground permitted the workmen to rest after being served hot coffee with the lunch they had selected and brought in with them.

"Drill doctor" benches and electricians' cupboards were distributed through the headings at convenient cross-cuts in order to save delays. Mine telephones were also installed at cross-cuts.

Enlargement in Hard Rock.—At East Portal the enlargement of the Railroad Tunnel center heading from 8 by 8 ft. to 16 by 24 ft. was accomplished for the most part in one operation by the ring shooting method. Rings of 27 holes each were drilled 4 ft. apart, the direction and spacing of the holes normal to the center line of the tunnel being controlled by steel templates attached to the drill housing or by offsets measured from the center line. Drilling was carried far in advance of the bench.

In shooting, the lower half of the excavation was kept about 16 ft. ahead of the upper ring. As a rule, a round consisted of two lower half rings, one without, and one with, delay electric exploders, followed by two top half rings with "delays". "Overbreak" in the railroad enlargement approximated 20%, while, in the Water Tunnel, it averaged 50 per cent.

The air shovel having a 1½-cu. yd. bucket with manganese steel teeth consistently averaged 24 ft. per day (two shifts) with a maximum of 53 ft. and maximum monthly progress of 1 002 ft. It moved 11 000 cu. yd. per month and remained underground 18 months without being overhauled.

In 1925, a crushing and screening plant was installed on the East Approach dump and 2½-in. ballast as well as sand and aggregates for concrete and "gunite" was produced from the tough granite in the eastern part of the tunnel. Altogether, more than 66 000 cu. yd. were treated at about 50 cents per yd., including cost of the plant, operation, and maintenance.

Gunite.—Through ground that would decrepitate, or air-slack, upon exposure, a coat of "gunite" (1 part of cement to 3 parts of sand), 1½ to 3 in. thick, served as an ideal protection. At about one-tenth the cost of timbering, it accomplished the same results providing there was no weight to support. It was particularly valuable over diagonal seams that might wash out and release blocks of rock after the air had softened the seam. Such spots would require a long stretch of timbering, whereas the gunite could be applied locally.

Beginning in 1925 gunite was used extensively covering thousands of square feet of rock surface at a cost for a 2-in. coat of 7 cents for labor plus 8 cents for material, or $8 to $14 per lin. ft. of tunnel as actually placed. About 20 000 sq. ft. was a good month's work. Much of the rock east of the apex was thus protected, only about a mile being left exposed.

Water Inflows.—Water was the greatest handicap encountered at East Portal. On February 18, 1925, a water seam was cut in the Water Tunnel 8 050 ft. in from East Portal.* This connected directly with Crater Lake, 1 300 ft. above, and produced 1 800 gal. per min. until silt from the lake bottom clogged the seam. Again, in October, 1925, water seams were encountered, which produced several hundred gallons per minute and materially retarded progress. As new seams were cut, the back seams would dry up, so that the headings, which were then on the down grade beyond the apex (Station 2536+54), required pumps.

The main seam 1 800 ft. below Ranch Creek was tapped on February 28, 1926. The "graveyard shift" coming in to muck out the heading hung up their electric light cluster and returned to Cross-Cut No. 10-E, 300 ft. back, for tools. When they returned to the heading it was noticed that the lights had apparently been moved. By this time the water, which had entered from below the muck-pile and had been slowly moving it with respect to the lights, burst through. It carried the pile 20 ft. down the tunnel and brought in 900 yd. of fine materials which were strewn for 200 ft. down the track.

The mucking machine and tools were moved to Cross-Cut No. 10 before the water reached the waists of the men, who then retreated to the apex 1 250 ft. away and installed pumps. Meanwhile, the water ran through the cross-cut and down to fill the Water Tunnel heading. Within 5 hours both headings were filled and water started to flow eastward over the apex through the Water Tunnel, the Railroad Tunnel having been dammed with sand-bags to protect enlargement work there. When the water reached Cross-Cut No. 8, it was diverted into the Railroad Tunnel east of the bench operations.

For a time a weir built at Cross-Cut No. 8 registered 2 500 gal. per min. and the tracks were covered with water, interfering with the electric locomotives; but within a week, the flow having subsided to 1 500 gal. per min., the headings were unwatered and work was resumed. In unwatering the headings, pumps and pipe were mounted on flat cars (Fig. 4) and backed toward the water, pumping under heads as high as 10 ft. From March 8 to April 8, the flow was 1 500 gal. per min.; in May, the flow decreased to about 600 gal. per min., which had to be pumped until the headings were holed through in the Water Tunnel and the benches completed in the railroad bore.

Other flows, totaling 450 gal. per min., were intercepted between Cross-Cuts Nos. 13 and 14. These were in looser rock and had the effect of softening the adjacent ground, thereby interfering not only with the holing through of the headings, but also with the excavation of the last 100 ft. of bench, which happened to come near the same location.

* "Construction Methods on the Moffat Tunnel," by R. H. Keays, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 92 (1928), p. 68.

At the completion of the tunnel a fairly constant flow of about 600 gal. of water per min. was leaving the tunnel *via* the West Portal drainage gutters and about 30 gal. per min. out of East Portal.

Operation at West Portal

General Plan.—At West Portal, the Water Tunnel functioned as a pioneer tunnel, advancing far ahead of the Railroad Tunnel enlargement so that heading, widening, and enlargement operations could be carried on simultaneously through the numerous cross-cuts. In this way as many as twenty attacks were made at different points at the same time, greatly expediting the work through the hazardous ground.

As at East Portal, all piping, wiring, and heading transportation was carried through the service tunnel, leaving the enlargement operations free of interference from these sources. The piping system included 7¾-in. casing for high-pressure air, 3½-in. water line, 12-in. ventilating line, and, in some places, pump discharge lines. The wiring provided for a 2 300-volt, 3-phase, 60-cycle, lead-covered cable supplying motor-generator sets at strategic points in the Water Tunnel; 250-volt direct-current wires for trolleys, mucking machines, and pumps; a 250-volt, 50-ampere "shooting" line; a 110-volt lighting line; and a telephone line.

Where pressures were encountered, top headings were followed by widenings for roof segments which, in turn, were followed by the excavation of the bench below the wall-plates either in one or two lifts. At some places west of the apex in rock that appeared solid, center headings were attempted and the Water Tunnel grades were accommodated to this 8-ft. change. However, such sections almost invariably required lining when opened up to the full size of the railroad bore. In changing from center to top headings, gradients as high as 5% were negotiated by the muck trains, but 2% was the maximum for machine mucking.

Bench Excavation.—The bench excavation presented the most difficult problems through the soft ground. Usually by the time the bench crew of fifteen men started work the large rock area had been exposed long enough to make it treacherous if it was not actually developing extreme pressure, tending to force in the side posts and allowing the roof to settle.

The cycle of operations on the bench ranged from 12 to 23 hours, as a result of which a "call shift" was instituted. The drilling of thirty to forty horizontal holes 20 ft. deep proved more satisfactory than drilling vertical down holes ahead in an attempt to save time.

Loss of Life.—During the early part of 1925 difficulties reached a crisis. At the same time that water was hindering progress at the East Portal, crushed, folded, and faulted rock was causing delays in the West Portal headings and widenings. Small cave-ins occurred in the headings and crushing wall-plates threatened to let down the roof timbers in the enlargement as unprecedented pressure developed.

One serious cave-in occurred in the Railroad Tunnel 175 ft. east of Cross-Cut No. 9 (2.3 miles from West Portal), on July 30, 1925, when six workmen of the day-shift timber crew were killed by 100 tons of rock that

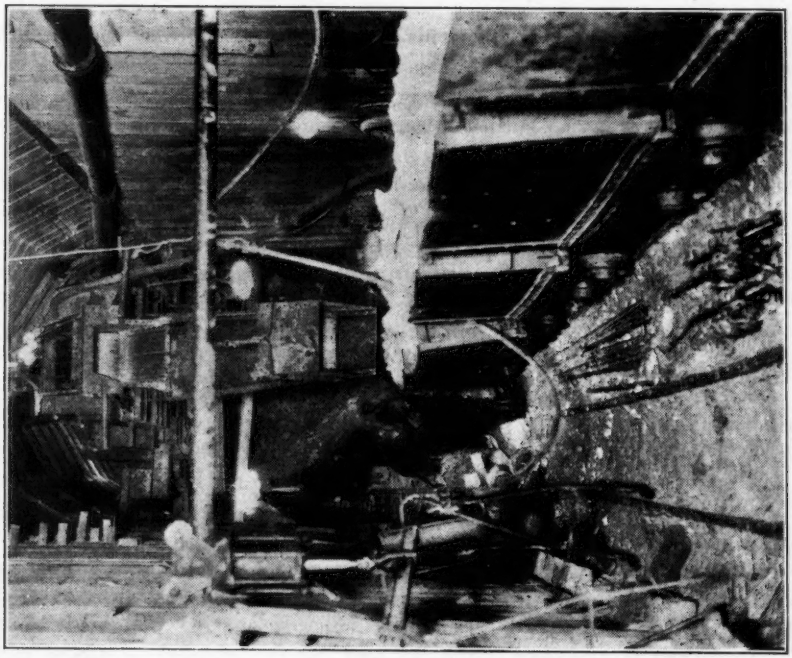


FIG. 5.—OPERATION OF LEWIS GIRDER DISCHARGING MUCK INTO CARS. AIR-LIFT CAR SWITCHER SHOWN IN FOREGROUND.

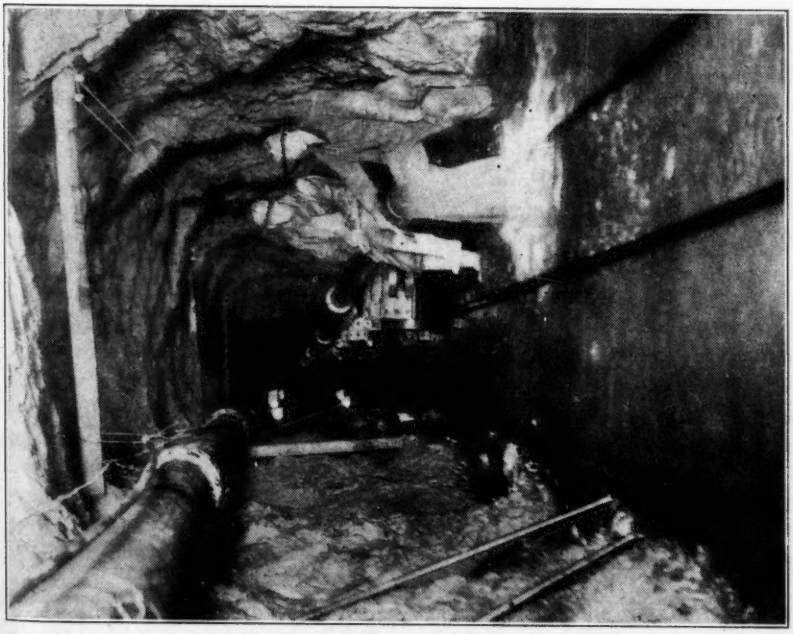


FIG. 4.—UNWATERING HEADING. PIPE LINE DISCHARGING AT APEX WHENCE WATER FLOWED BY GRAVITY TO PORTAL.

fell without warning from the north side of the roof as timbers were being placed. Characteristic heroism was displayed by the miners who volunteered to a man for the rescue work which was organized in case any of the men were buried alive. The coroner returned a verdict of "unavoidable accident".

In all, twenty-eight men made the supreme sacrifice in putting through the Moffat Tunnel—a remarkable record considering the hazardous nature of the formations encountered.

Lewis Traveling Cantilever Girder.—In order to cope with the emergency the first Lewis traveling cantilever girder, originated by George Lewis, General Manager, was installed at the West Portal bench near Cross-Cut No. 2 in March, 1925. This invention (Figs. 5 and 6) had numerous advantages, foremost among which was mobility. Designed to support the roof timbers and wall-plates (as a cantilever resting on, and extending back from, the heading) while the bench was being excavated and posts set, this machine successfully held the weight and increased the speed of progress. It is valuable in soft-ground tunnels of large size and can be used for either heading and bench or pioneer tunnel operations.

Four of these girders were used on lined sections of the Moffat Tunnel extending two miles or more, and their effectiveness has been tested through soft running ground of slicken-sided schist and gneiss as well as through blocky granite. On one occasion, Girder No. 1 caught a cave-in, protecting the workmen beneath. After tons of muck had been removed, the long girders, which had been bent downward several inches at the end, returned to their original alignment.

Girder No. 2 installed in October, 1926, was equipped with a conveyor belt, 30 in. wide, operating at 30 ft. per min. by means of a 50-h.p. electric motor. Being reversible, this conveyor could carry muck from the heading back through the center of the girder and discharge it through a 14-ft. chute into 4-yd. cars behind the bench (Fig. 5) or it could carry timbers and supplies from the bench ahead to the headings. The other three girders were without conveyors inasmuch as they were used for benches where widening had been handled through cross-cuts ahead. A description of Girder No. 2 (Fig. 6), therefore, will cover the others.

Two plate girders, 42 in. deep by 65 ft. long, spaced 6 ft. apart with rigid cross-braces of structural angles, which also served as the frame for the conveyor rolls, constituted the backbone of the machine. Cross-arms extending at right angles to the center line of the girder and supported beneath it by steel stirrups could be moved out under the wall-plates and jacked tightly up against them, using 15-ton screw-jacks built with the hanger and operating through a lever and rocker-arm. There were twelve cross-arms at the rear, or bench end, of the girder to support the roof and four at the forward end to keep it in place against the up-thrust when the load was applied at the rear end. Jacks against the roof were sometimes used for the same purpose.

In addition to the twelve cantilever arms beneath the wall-plate there were two that rested upon the top of the wall-plate, converting the cantilever into

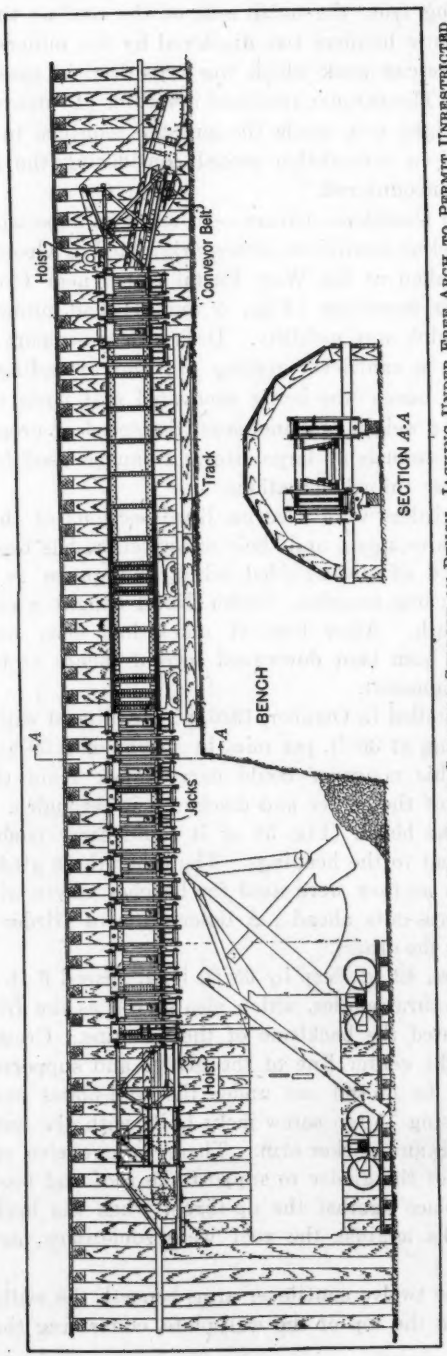


FIG. 6.—LEWIS TRAVELING CANTILEVER GIRDER: ELEVATION SHOWING SUPPORT ON BENCH, JACKING UNDER FREE ARCH TO PERMIT UNRESTRICTED MUCKING, AND CONVEYOR SYSTEM.

a beam and increasing its capacity under load. The entire machine rode on two pairs of steel dollies with ball-and-socket joints, which traveled along a track of steel channels on 12 by 12-in. timbers. Web stiffeners reinforced the ½-in. web-plates above the dollies.

In making a set-up, four forward cross-arms were first rolled out under the wall-plate, wedged with filler blocks, and put under pressure by the jacks, thus throwing the weight on the front dollies. Overhead jacks or rear cross-arms were next tightened until the girder took the entire roof load so that the temporary short posts could be removed. The girder, as designed, weighed about 200 000 lb. Blocks between the side of the girder and the wall-plates counteracted any inward movement due to unequalized roof and side-wall loads.

With the girder in place the temporary short posts under the wall-plates at the bench were removed, and the bench was drilled, shot, and mucked while the girder supported the roof load. The average progress per shift varied from 12 to 24 ft. depending on the ground.

Following this, 12 by 12-in. or 12 by 18-in. posts of Oregon fir were pulled up into plumb position by an air hoist attached to the end of the girder (Fig. 6) and, like the lights and wiring, traveling with it. When these had been wedged in place beneath the wall-plates, the cross-arms were released and pulled back under the girder so that the entire machine could be moved ahead by means of an electric hoist working on a cable attached to the timbering ahead. Blowers mounted on the girders were useful in ventilating the heading.

Steel Sets.—Where heavy timber lining* was insufficient, steel sets with arches and inverts were designed to carry the loads as roughly determined by the failure of dimension timber in the headings. The first, or lighter, steel sets consisted of 24-in. by 80-lb. I-beam posts, three 20-in. by 65.4-lb. I-beam roof segments, and three 18-in. by 54.7-lb. I-beam invert members (Fig. 7) placed on 3-ft. centers so that additional sets could be "jumped in" between if necessary. This had to be done in a few instances. When it became evident that heavier reinforcement was needed to hold the ground until concrete was in place, a "heavy steel set" was designed consisting of two 20-in. by 113-lb. girder beam posts, three 14-in. by 100-lb. H-beam arch members, and three 12-in. by 79-lb. H-beam invert segments. The 10-in. channel collar braces between sets were encased in concrete as soon as bolted to prevent twisting as the ground flowed, and reinforced pre-cast foot-blocks were used to prevent undue settlement of the posts (Fig. 7).

Roof arches and 24-in. I-beam wall-plates were first placed, followed by the posts which were set behind the Lewis girder. Between the posts at a distance of about 8 ft. above the floor 12 by 18-in. wooden spreaders were wedged until inverts were fabricated. Some failures occurred in these spreaders and, in a few instances, the ends of the timbers pushed in the flanges of the steel posts more than an inch causing the web to buckle and necessitating

* "Construction Methods on the Moffat Tunnel," by R. H. Keays, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 92 (1928), p. 95.

replacement of the post. In places, the floor squeezed up several inches a day, requiring repeated excavation before being finally filled with concrete.

Costs of timber lining and of steel lining are given in Appendices III and IV.

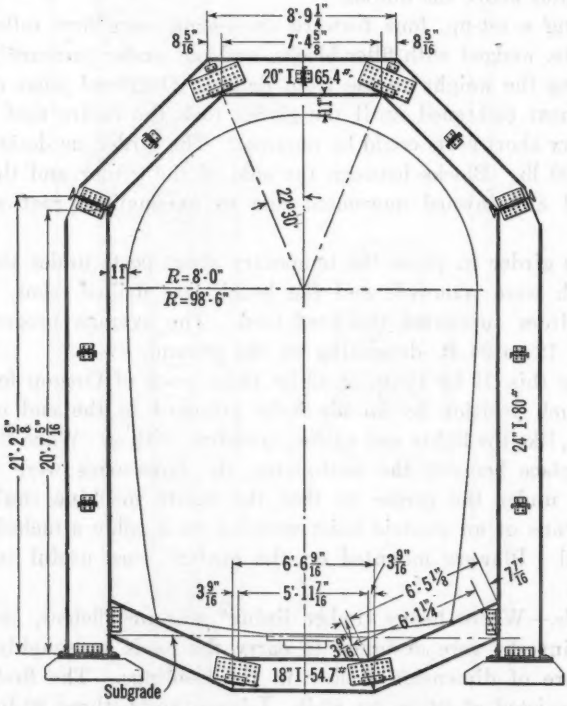


FIG. 7.—STEEL SET FOR TUNNEL LINING—LIGHT TYPE.

Concrete Lining.—To minimize this squeezing, which increased with exposure of the soft rock to air and moisture up to a point where arching created a pseudo-equilibrium, concrete was placed in the inverts as soon as possible after the steel. Reinforcement of 30-lb. rails and 1-in. bars was placed between the 5½-ton steel sets so that the entire lining was consolidated. Folding wooden forms in 4-ft. sections, lined with sheet zinc and suspended by block and tackle from the timber spreaders, were held down by bracing until the concrete set, with the result that none of the concrete was ruptured during the early setting stages. Cement which gave early compressive strength of more than 2 000 lb. per sq. in. in 48 hours and 4 500 lb. in 7 days was used.

In one of its reports to the Moffat Tunnel Commission, the Board of Consulting Engineers called attention to the tendency of the weak rock, if held securely, to reach a constant distributed bearing, insuring permanent safety. Instrumental observations made on the lining to determine the movement, if any, had shown this to be the case.

Following the lining of the inverts, traveling steel forms, mounted on a timber "jumbo" carriage, were moved along the tracks on the finished invert.

When jacked into position the bottom of these forms was fastened to bolts previously embedded in the upper shoulder of the invert near the keyway joint. In carrying the invert up to this joint, 4 ft. above the center-drain flow line, reinforcing rods were left protruding upward so that the steel for the side-walls and arches could be tied to it.

Material for the sides and arches was fed from concrete guns through 8-in. steel pipe to the crown of the arch whence it flowed behind the forms, being assisted by workmen with shovels or by hammering with one-man air hammers on the outside of the forms—a method producing dense concrete. The forms were supplied with inspection windows. Both stationary and moving concrete gun set-ups were used. In the former a concrete mixer was mounted on a platform above the tracks in a completed section of the Railroad Tunnel and supplied with cement, sand, and gravel by endless bucket conveyors located at the side of the track. Concrete mixed 1½ min. in the proportions by volume of 1 : 2 : 3 (water-cement ratio, 0.8 to 1.0) was then poured by gravity into the concrete gun below and shot under 80 to 110-lb. pressure through the 8-in. pipe line to the forms.

It was found that short pipes gave the best results inasmuch as concrete shot a distance of more than 100 ft. had a tendency to segregate, some of the cement blowing out in vapor. To obtain a 4-in. slump in the forms required 6 to 7-in. slump at the mixer. Hence, a concrete gun was mounted to one side of the track on the traveling "jumbo" that carried the forms and was supplied with concrete from the central mixing plant by special side-dump bodies mounted on flat cars.

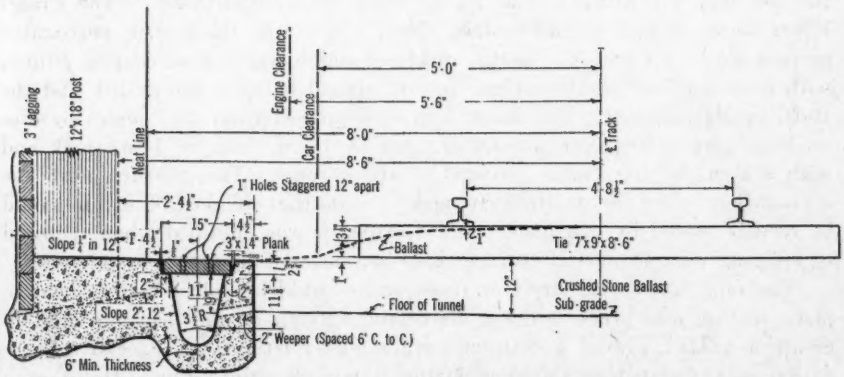


FIG. 8.—DRAINAGE GUTTERS FOR RAILROAD TUNNEL, CROSS-SECTION SHOWING RELATION TO WALL AND TRACK.

In all, 916 lin. ft. of steel lining was buried in concrete and a total of 12 000 yd. of concrete lining was used, including that around the wooden sets. The thickness of the concrete ranged from 1 to 5 ft. Throughout the timbered sections allowance was made for future lining of concrete by setting the posts back. The far-sighted policy of concreting the sections showing greatest need each year will, if followed out consistently, eventually result in a complete permanent lining—a desirable and, in the long run, an economical condition. The cost of concrete lining will be found in Appendix IV.

Drains.—Concrete gutters through the timbered sections (Fig. 8) served the dual purpose of protecting the timbers from the deteriorating and undermining effects of water and of anchoring the posts against "kicking in". A capacity of about 3 000 gal. per min. was provided in the gutters, work on which was begun in April, 1926, and carried on by separate crews following the lining operations.

Center drains through the concrete-lined sections were connected with the concrete side gutters by transitions. Tile drains, made to order, were laid with open joints along the south side of the tunnel through the solid rock sections east of the apex. These were entirely covered with ballast.

Spark Protection.—A canopy 10 ft. in width, of 1-in. by 4-in. California redwood, tongued and grooved, was nailed to the roof segments through the timbered sections to prevent live sparks from lodging in the cracks and starting fires. This type of fire protection has been used on timbered tunnels along the Moffat Railroad.

Refuge Bays.—Twenty-one niches, or refuge bays, were left in the south wall of the railroad tunnel at cross-cuts for the convenience of maintenance crews.

Water Tunnel

Steel Plate.—As the excavation in the Railroad Tunnel progressed the timbering in the Water Tunnel, 62 ft. away, showed movement and distress, making it imperative that this section be protected from cave-ins. Accordingly, 1 485 ft. of the water bore east of Cross-Cut No. 9-W was enlarged to the size that will ultimately be required for the transportation of the Fraser River water supply to the eastern slope. This was lined with corrugated pressed steel liner plates (Fig. 9), inside of which was a 4-in. coat of gunite, with allowance for additional concrete if needed between the gunite and the 10-ft. inside diameter. The liners were cold-pressed from steel plates by dies made to give twelve segments 36 in. long by 16 in. wide by ¼ in. thick and with a 2-in. bolting flange rounded at the corners. This feature, with the corrugations, gave the required strength to units that weighed 50 lb. and could be readily placed by one man. Added strength was obtained when required by splicing ¼-in. segmental stiffener bars between the flanges.

The fact that a full circle of these plates could be bolted and wedged in place to line and grade without disturbing enough of the heavy ground to create a hazard, proved a distinct advantage. Furthermore, excessive excavation was obviated as the steel virtually became reinforcing. Holes were provided at intervals of a few feet so that the packing behind the steel plates could be grouted under pressure. Having thus taken the weight of the ground on this circular steel lining a protection of gunite (1 part cement to 2 or 3 parts sand) was placed to a uniform depth of approximately 4 in.

Loads estimated at 129 000 lb. per lin. ft. were supported by this type of lining without deformation in the steel or cracking in the cement.

Future Use.—The Moffat Tunnel Law made it mandatory upon the Commission to provide for transportation of water from the Colorado River Basin on the western slope to the dry eastern slope; the purpose being to increase

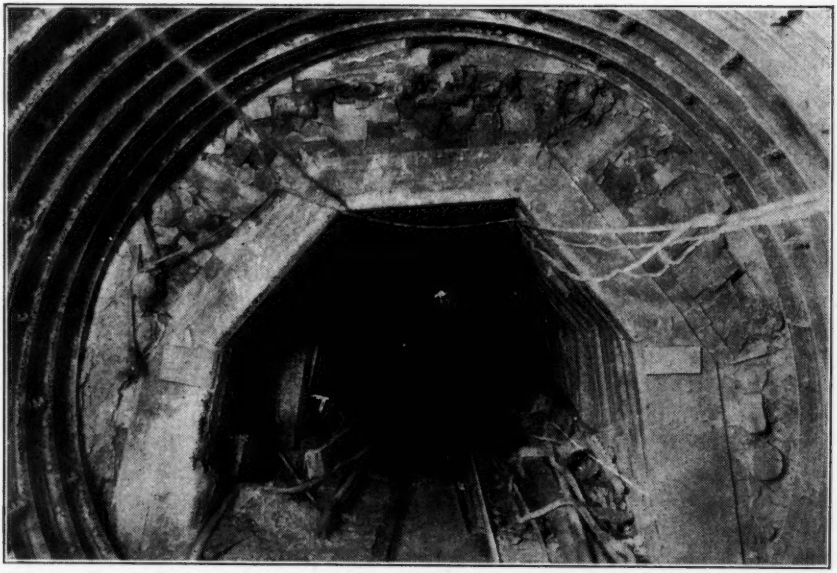


FIG. 9.—WATER TUNNEL, SHOWING WOOD TIMBER LINING BEING REPLACED BY CORRUGATED PRESSED STEEL LINER PLATES.

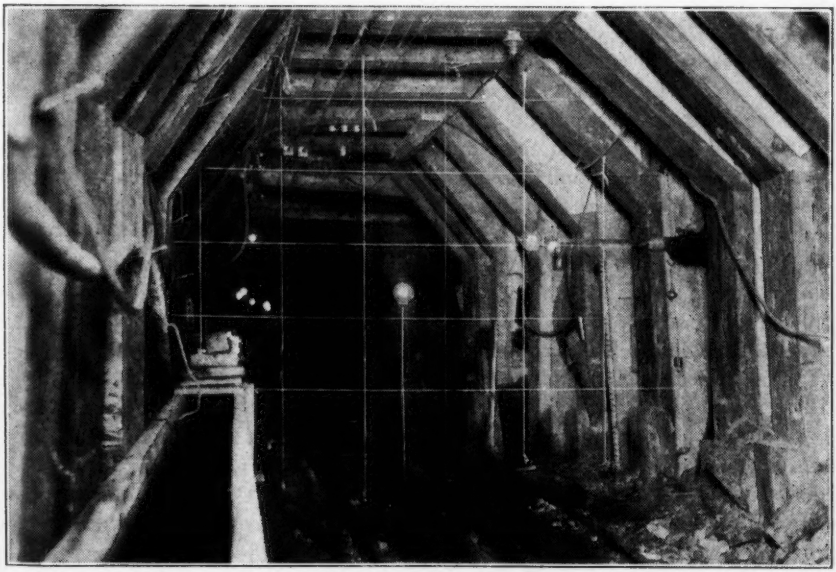


FIG. 10.—AIR-MEASURING STATION IN WATER TUNNEL—DETERMINING MEAN VELOCITY IN CROSS-SECTION.

the available water supply for the City of Denver. Therefore, plans were made for a parallel bore as close as practical to the Railroad Tunnel that would serve as a service tunnel to speed up construction and later could be enlarged to carry 1 000 cu. ft. per. sec. of water—the capacity required to divert economically 100 000 acre-ft., the mean run-off from the 97 sq. miles of the Fraser River Diversion Project upon which the City of Denver had filed.

To carry water, the tunnel must be lined throughout the western end from the intake shaft at Station 2696 + 25* to the apex at Station 2536 + 81 as well as through approximately 1 500 ft. of scattered seamy sections east of the apex. The remaining 12 200 ft. east of the apex is through solid granite having an average area of 100 sq. ft. (8 by 8 ft. + 50% overbreak) which would carry about 500 sec-ft. until need for the full 1 000-sec-ft. capacity necessitated enlargement or lining, or both.

According to plans contemplated, water collected by a canal system from the Fraser River, Vasquez Creek, St. Louis Creek, Ranch Creek, and tributary streams would reach the 10-ft., circular Water Tunnel through a 209-ft. vertical shaft, 12 ft. in diameter at Cross-Cut No. 3 (2 700 ft. east of West Portal). The water would be forced under pressure (gravity) over the apex, Elevation 9 244, and thence down grade to East Portal, Elevation 9 204, the hydraulic gradient being 0.004. From East Portal a short canal could deliver the water into South Boulder Creek which flows through rocky box canyons to Eldorado Springs on the edge of the plains, 15 miles northwest of Denver, and 500 ft. higher in elevation. This point is also above the head-gates of numerous irrigation ditches serving territory adjacent to the city.

Ventilation

Air flow tests made at the Moffat Tunnel (Fig. 10) to determine the coefficients of friction of the lining preparatory to deciding upon the specifications for the blowers, furnish valuable data on the relation between flows through tunnels of railroad size and through mine tunnels. The information available on the former is fragmentary, that on the latter, voluminous; so that the rare opportunity offered by the Moffat Tunnel measurements to compare the two simultaneously as the same volume of air was forced through the 8 by 8-ft. and the 16 by 24-ft. bores, will be useful in converting the extensive experimental data on small mine airways to the solution of railway tunnel ventilation problems.

The determination of the power requirements for mechanically ventilating the tunnel, necessitated estimating both the volume of flow desired and the pressure required to circulate this quantity of air through the tunnel. In both these estimates a large number of variables are involved.

Estimating Volume of Flow.—Steam operation having been determined upon by the lessee of the tunnel, the primary purpose of ventilation would be to dilute and remove the noxious gases produced by combustion of coal in the engine. The requirements are based on diluting the carbon monoxide of the locomotive exhaust gas to a proportion that would have a negligible effect on man.

* "Construction Methods on the Moffat Tunnel," by R. H. Keays, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 92 (1928), p. 66, Fig. 3.

In arriving at this quantity the following assumptions and calculations have been made:

Maximum train—locomotive and 50 cars: 2 500 tons.
Maximum tractive resistance at 24 lb. per ton on 0.9% grade: 60 000 lb.
Train speed: 10 miles per hour.
Maximum tractive power: 880 000 ft-lb. per sec.
Necessary coal consumption based on test: 0.73 lb. per sec.
Air for combustion at 25 lb. of flue gas per lb. of coal: 18.3 lb. per sec.
Air for dilution, assuming 1% CO in gas (based on tests), diluted to 5 parts per 10 000 (4 parts per 10 000 considered safe limit for 1 hour exposure): 330 lb. per sec.
Quantity required at 0.055 lb. per cu. ft., average weight of air at the tunnel: 360 000 cu. ft. per min.

Actually, this result is considered a maximum in that average maximum values have been assumed throughout.

Estimating Pressure Requirements.—The pressure required to cause any particular volume of air to flow through an airway, such as the Moffat Tunnel, depends on the dimensions, velocity of flow, density of air, and the physical characteristics of the airway. The latter must be handled in calculations by the use of empirical numbers which are based on experimental determinations and designated as friction factors or coefficients of friction.

Friction factors for somewhat similar types of mine airways, 40 to 60 sq. ft. in area, were available from experimental work of the U. S. Bureau of Mines, but their application to such large airways (400 sq. ft.) was extremely uncertain as was also the available evidence concerning the effect of area on friction factors, which was based on small pipe experiments. In view of these facts and the lack of preciseness of visual comparison of rough rock surfaces, it was considered necessary to determine friction factors for typical sections of the partly completed tunnel.

These tests indicated that a maximum average value of the friction factor for air weighing 0.055 lb. per cu. ft. would be about 0.000 000 0050.

Tunnel Resistance.—Using the formula,

$$R = \frac{K P L Q^2}{A^3} \qquad (1)$$

the resistance of the clear tunnel was calculated as follows:

0.000 000 005	=	assumed friction factor, K.
$\times$ 80	=	average perimeter, P, of tunnel sections, in square feet.
$\times$ 32 250	=	length of tunnel, L, in feet.
$\times$ $(360\,000)^2$	=	square of estimated quantity of flow, Q, in cubic feet per minute.
$\div$ $(400)^3$	=	cube of average area, A, in square feet.
= 26.16	=	resistance, R, in pounds per square foot.
= 5.03	=	resistance of flow, $\frac{R}{5.2}$, in inches of water.

In like manner, the resistance of the fan ducts leading to the tunnel was estimated as approximately 0.02 in. of water.

Resistance of Standing Train.—No data being available for the resistance caused by moving trains, the resistance of a standing train was first calcu-

lated from experimental data and from rational assumptions. Then the difference in results was used as a criterion in arriving at the resistance of a moving train calculated by the latter method.

With coal mine cars occupying about 20% of the total area, U. S. Bureau of Mines experiments gave an end resistance of 0.45 of the velocity pressure in the clear airway for the first car and 0.15 of the velocity pressure for the side resistance of each additional car when closely coupled. Tests on single cars of different areas indicated that the resistance varied directly as the square of the ratios of car area to airway area. Assuming that the locomotive has an effect equal to 2 cars, that the average train will comprise 50 cars, and that the cars are approximately 9 by 14 ft. in section and thus occupy about 31% of the tunnel area, the total train resistance may be calculated as:

$$\left(\frac{31}{20}\right)^2 \times [0.45 + (52 \times 0.15)] = 19.8 \text{ velocity pressures}$$

At 900 ft. per min. the velocity pressure is 0.037 in. of water for air weighing 0.055 lb. per cu. ft.; therefore, the train resistance is:

$$19.8 \times 0.037 = 0.734 \text{ in. of water.}$$

The resistance of the standing train may also be calculated by considering it made up of two components: The impact against the end, destroying $\frac{125}{400}$ of the velocity pressure, and the actual increase in resistance due to the air moving at higher velocity through the reduced section. The end resistance is then $\frac{125}{400} \times 0.037 = 0.012$ in. Assume a friction factor of 0.000 000 005; and a length of 2 000 ft., with the area reduced to 275 sq. ft., the perimeter increased to 110 ft., and the velocity increased to 1 319 ft. per min. Then the resistance through the reduced section is calculated as 1.318 in. of water. The tunnel resistance without the train is $\frac{2\,000}{32\,250} \times 5.03 = 0.312$ in., therefore the increase will be $1.318 - 0.312 = 1.006$ in. The total train resistance is then $1.006 + 0.012 = 1.018$ in., a result which is approximately 40% higher than that calculated from experimental data.

Resistance of Train Moving Against the Air Flow.—Since rational computations approximate experimental results for standing trains, it is felt that a sufficiently accurate calculation of the resistance caused by the train moving against the air flow at 10 miles per hour may be made. In this case the train velocity of 880 ft. per min. is added to the tunnel air velocity of 900 ft. per min., giving a relative velocity of 1 780 ft. per min. for the air impacting on the train, for which the velocity pressure is 0.081 in. of water. The end resistance is then $\frac{125}{400} \times 0.081 = 0.025$ in.

The velocity through the reduced section of 275 sq. ft. is 1 319 ft. per min., while the relative velocity along the train is 880 ft. per min. greater, or 2 199 ft. per min. At 1 319 ft. per min., the resistance of 2 000 ft. of this

reduced section is calculated as 1.32 in., while at 2 199 ft. per min., it is 3.67 in. (See Equation (1).) Considering these values as applying in the ratios of the perimeters involved, the total side resistance would be:

$$\frac{73}{110} \times 1.32 + \frac{37}{110} \times 3.67 = 2.11 \text{ in.}$$

For this section the normal resistance without the train is 0.31 in. and the increase due to the train is 1.80 in. This, added to the end resistance of 0.025, gives a total increase in resistance due to the moving train of 1.83 in. of water. Assuming that the values thus calculated are 40% high, the train resistance is 1.31 in.

Total Resistance to Flow.—On the basis of a flow of 360 000 cu. ft. per min., the total resistance has been calculated as follows: 5.03 in. for the tunnel proper; 0.02 in. for the snowshed and fan connection; and 1.31 in. for a train moving 10 miles per hour against the flow, or a total of 6.36 in. of water.

For any other flow the combined resistance may be computed by means of the equation,

$$R = 0.49 \left(\frac{Q}{100\,000}\right)^2$$

in which, R equals resistance, in inches, of water; and Q, quantity of air, in cubic feet per minute.

Natural Draft.—Immediately after "holing through" the Water Tunnel the flow of air from west to east was measured and found to be 50 000 cu. ft. per min., corroborating the barometric readings made at the two portals.

At these points situated on the opposite sides of a mountain, pressures are rarely the same, due to differences in atmospheric conditions in the columns of air above. Barometric records have been kept at both portals since March, 1925, and these, corrected for difference in elevation, indicate that a strong natural draft from west to east predominates during the winter months, and a variable intermittent draft with frequent reversals, during the remainder of the year.

The average maximum draft west to east may be taken as about 0.3 in. of mercury (4.1 in. of water), a value that is exceeded but four or five times a year and only for relatively short periods. The maximum recorded draft has been 0.7 in. of mercury (9.5 in. of water). The average maximum draft east to west may be taken as about 0.05 in. of mercury (0.7 in. of water), a value that is rarely exceeded and but infrequently attained. The maximum recorded draft has been 0.15 in. of mercury (2.0 in. of water).

The other minor components that might be considered are the difference in weights of the air column inside the tunnel and that at the West Portal through the same elevation; and the effect of wind pressure on the portals. The former amounts to but 0.11 in. for an average difference of 40° in temperature between the inside and the outside of the tunnel, an extreme winter condition of rather short duration. Gales are frequent at both portals during the winter months, blowing against the West Portal and down over the East Portal, the latter probably producing some suction effect.

Thus, natural draft pressures tend to increase the demand upon the ventilating equipment, at least in the winter when strong adverse natural draft has to be contended with in ventilating against the heavily loaded trains moving from west to east up the 0.9 and 0.8% grades west of the apex.

Air Flow Tests to Determine Friction Factors

Air flow tests were made in March, 1927, shortly after the "holing through" of the Water Tunnel, while the main headings of the Railroad Tunnel were about 1 000 ft. apart and the enlargement operations were about 4 800 ft. apart (Fig. 11). Conditions were not ideal for experimental precision inasmuch as tunneling operations were going on day and night. However, about 20% of the observations were free from interference.

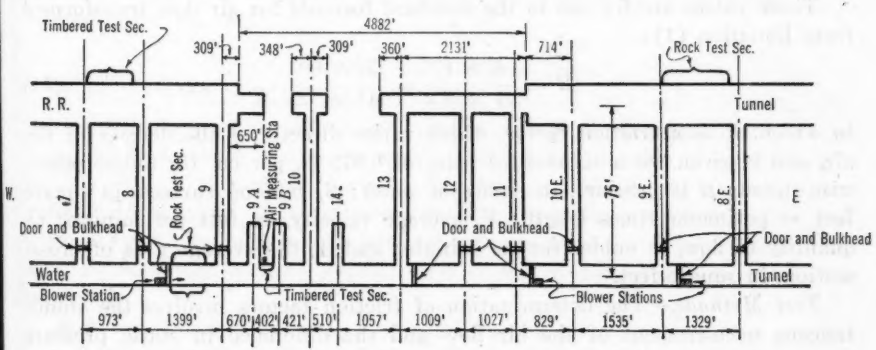


FIG. 11.—AIR FLOW FRICTION TESTS: LAYOUT OF SECTIONS, BLOWERS, AND DOORS.

Velocities were restricted by the high resistance to air flow through the 8 by 8-ft. Water Tunnel and cross-connections between the uncompleted sections of the Railroad Tunnel so that, although this arrangement insured the passage of all air through the gauging station in the Water Tunnel, it kept the maximum velocity through the Railroad Tunnel section down to about 80 ft. per min., equivalent to a flow of 35 000 cu. ft. per min.

Scope of Test Work.—Four sections (Fig. 11) were selected for tests: (1) A rock section of the Railroad Tunnel between Cross-Cuts Nos. 8-E and 9-E; (2) a timbered section of the Railroad Tunnel between Cross-Cuts Nos. 8-W and 7-W; (3) a timbered section of the Water Tunnel between Cross-Cuts Nos. 9-A and 9-B; and (4) a rock section of the Water Tunnel between Cross-Cuts Nos. 9-W and 8-W.

General Statement of Test Results.—Although individual test results varied widely, especially for short-period tests on the Railroad Tunnel sections, where very low velocities and minute pressure differences were involved, the averages of a large number of test observations seem quite reliable and check almost exactly previous estimates extrapolated from other experimental data. As a result of these tests and their comparison with available data, the friction factors shown in Table 1 were selected as applicable to such large airways.

TABLE 1.—VALUES OF FRICTION FACTOR, K, IN THE MOFFAT TUNNEL.

	FOR AIR WEIGHING:	
	0.055 lb. per cu. ft.*	0.075 lb. per cu. ft. (standard density).
Unlined rock	0.0000000050	0.0000000068
Timbered on 4 to 5-ft. centers	0.0000000045	0.0000000061
Timbered on 3-ft. centers	0.0000000040	0.0000000055
Timbered on 2-ft. centers	0.0000000035	0.0000000048
Timbered skin-tight	0.0000000015	0.0000000020
Smooth-lined	0.0000000010	0.0000000014

* Estimated average maximum weight of air in Moffat Tunnel.

These values are for use in the standard formula for air flow transformed from Equation (1):

$$R = \frac{K S V^2}{A \times 5.2} = \frac{K S Q^2}{A^3 \times 5.2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1a)$$

in which, K is a friction factor, which varies directly as the density of the air, and is given for a standard density of 0.075 lb. per cu. ft., unless otherwise stated; R is pressure, in inches of water; S, rubbing surface, in square feet = perimeter times length; V, average velocity, in feet per minute; Q, quantity of flow, in cubic feet per minute; and A, the average area of cross-section, in square feet.

Test Methods.—The determination of friction factors involves the simultaneous measurement of the air flow and the difference in static pressure between the two ends of the section selected for test, together with sufficient observations of barometric pressure and temperatures for determining the weight of the air. From these data and the dimensions of the airway, values of the friction factor, K, are computed from Equation (1*a*).

Tests on three of the four sections required the sealing of a door on a main traffic artery, therefore, simultaneous observations were required at three widely separated points, at any one of which test observations might be, and were, interrupted for varying time intervals. Continuous observations were made at all three points for a period of several hours at a time; the results were then compared on a time basis and all those showing interference (about 80%) were discarded before computations were made. The remaining 20% were usually scattered over relatively short periods.

Under the test conditions, air measurement by calibrated anemometers was considered more practical than Pitot tube, or velocity-pressure, methods. Air flow was measured at a timber set in a heavily timbered section of the Water Tunnel just east of Cross-Cut No. 9-A. Preliminary traversing, as shown in Fig. 10, determined the relation of the mean velocity to the velocity at the center of the section and the latter only was measured during pressure-loss tests. A few center velocities, measured simultaneously by anemometer and Pitot tube, were about 2% higher by the latter method, which, considering the difference in areas involved, is ample assurance that the anemometer calibrations used were substantially correct.

Standard U. S. Bureau of Mines static tubes were set up at each end of the test section and pressures from these were communicated to a pressure gauge, near the middle of the section, through 100-ft. lengths of small (⅜-in. inside diameter) pneumatic hose connected by pure gum rubber tubing.

Wahlen differential pressure gauges were used to measure the difference in the pressures communicated through the tubings, to an accuracy of 0.0001 in. of alcohol. Temperatures in the tunnel were almost absolutely constant at any one point, but varied from 58° to 65° Fahr., in the test zone. (Rock temperatures had previously been found to range from 60° near the portals to 68° Fahr., near the center.)

Cross-sectional areas and perimeters were obtained from the graphical records for all sections except the Water Tunnel timbered section which was measured for the test.

Tunnel Conditions for Tests

Railroad Tunnel Rock Sections.—Test sections, 954 and 1 200 ft. long, through blocky granite of average roughness with some gradual area variations and ballasted floor gave an average friction factor value of 0.000 000 0070 for velocities of 57 to 83 ft. per min. (air density, 0.075 lb. per cu. ft.). Compared with previous tests on 60-sq. ft. airways having a friction factor of 0.000 000 0092, an increase of 24% corresponds to the 1:7 increase in area. The low velocities in these large airways do not appear to increase the factor materially.

Railroad Tunnel Timbered Sections.—In the sections tested, the timbers were in almost perfect alignment, but the spacings varied irregularly as they do throughout all timbered sections of the tunnel. Expressed in terms of length of section: Three sections, totaling 12%, were timbered skin-tight; six sections, totaling 25%, on 2-ft. centers; six sections, totaling 47%, on 3-ft. centers; and two sections, totaling 16%, on 4-ft. centers. Fig. 12(*a*) shows a typical cross-section.

Four tests on a 890-ft. section, at velocities of 74 to 81 ft. per min., were for 9 to 13 min. duration and were expected to yield very consistent results. The computed results, however, were somewhat erratic and their averages of 0.000 000 0053, although roughly checked by later observations at lower velocities, may be somewhat high.

Three variables are involved in the comparison of this with results secured at high velocities in small timbered airways, namely, the effects of area, of timber spacing, and of low-velocity flow. The tests on the Railroad Tunnel rock section indicate that the latter is negligible; and mine-airways data indicate a factor, K, of 0.000 000 0070 with timbers spaced on 5.3-ft. centers, so there is an indicated reduction in K of about 32% for the effects of area and decreased spacing combined.

Water Tunnel Timbered Section.—In this section, between Cross-Cuts Nos. 9-A and 9-B, the 12-in. faces of the timbers were in good alignment with but few irregularities and small offsets. The timbers were spaced 18 and 24 in., center to center, in alternate blocks, about one-third the length involved being timbered on 18-in. centers and two-thirds on 24-in. centers.

The floor was of loose muck with the track ties exposed throughout and with the bottom segments exposed for about one-third the length on one side. A water flume, approximately 1.0 by 1.1 ft., occupied part of one side of the section, resting on the floor at the east end, with the space beneath filled with pipes and blocking at the west end. Areas were taken inside the upper braces with no allowance for the spaces between these braces and the caps, since tests have shown that no appreciable flow occurs through such small openings. The shape of the section is shown in Fig. 12(*b*); and the type of surfaces in Fig. 10.

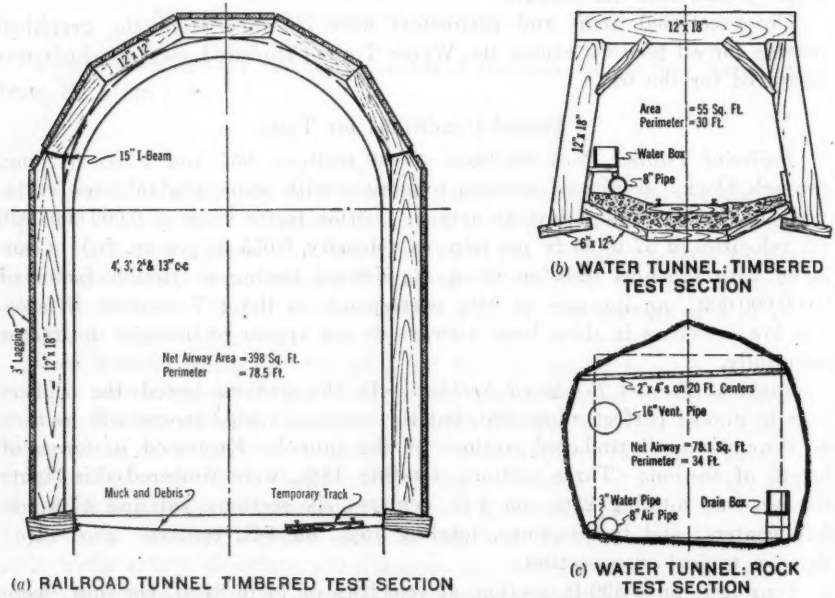


FIG. 12.—AIR FLOW FRICTION TESTS: TYPICAL CROSS-SECTIONS OF TUNNELS USED FOR MEASUREMENTS.

Since all observations were under sight control, 2-min. tests yielded quite consistent results even on different, but overlapping, test sections and in spite of the interference resulting from working on a small main traffic artery. Six tests on six different sections, 310 to 320 ft. in length, gave an average factor, K, of 0.000 000 0053 at velocities of 600 to 700 ft. per min. At velocities of 340 to 400 ft. per min., four tests on two sections gave an average factor of 0.000 000 0051.

Later tests at low velocities, with a small amount of timber storage in the test section, gave factors as high as 0.000 000 0073 at a velocity of 171 ft. per min. At 264 ft. per min., the factor was 0.000 000 0057 and, making some allowance for the added obstructions present, it seems that, for this section, a break in the law of flow occurs at a velocity of about 250 ft. per min.

An average value of K of 0.000 000 0052 is indicated for this section for velocities greater than 250 ft. per min. For a quite similar section tim-

bered on 5.3-ft. centers, a factor of 0.000 000 0080 has been determined in mine airways. There is thus an indicated decrease of about 35% as between 5.3 and 1.8-ft. spacing in airways of this size.

Water Tunnel Rock Section.—In this section, between Cross-Cuts Nos. 8-W and 9-W, the rock surfaces were more irregular and the area gradations were more abrupt and of greater extent than average conditions in railroad tunnel sections; and not only was the floor much rougher, but the section had a number of obstructions, including a fan pipe, 14 in. in diameter, with flanged joints, a water-flume and blocking, a 3-in. water pipe, an 8-in. air pipe, and 4-in. cross-bars on 20-ft. centers near the roof. A representative section is shown in Fig. 12(*c*).

For velocities greater than 250 ft. per min., very consistent results were obtained, the average for eight tests being 0.000 000 0148. Tests by the U. S. Bureau of Mines on a quite similar mine airway, but without obstructions present, would, after increasing the factor by about 0.000 000 005, 0.000 000 0010, and 0.000 000 0010 for the fan-pipe, cross-bars and water-flume, respectively, give the combined factor of 0.000 000 0140 for a similarly obstructed airway, a value which agrees quite closely with that determined for this test section.

A few tests at velocities less than 250 ft. per min. showed a rapid increase in the factor, K, as the velocity was lowered.

Evaluation of Test Results

Effect of Area on Friction Factor.—According to the laws of fluid flow, the friction factor is the same for the same values of the Reynolds number —the product of the weight of air, diameter of passage, and velocity divided by the viscosity; and experiments show that, for airways of a degree of roughness similar to the Moffat Tunnel, the factor is constant for all values of the Reynolds number, or for all velocities ordinarily encountered. However, this law assumes that variations in area are accompanied by a geometrically similar variation in surfaces, and does not apply exactly where variations in area are accompanied by a constant degree of roughness of surfaces.

In estimating factors for Moffat Tunnel sections, approximately 400 sq. ft. in area, from test data on 40 to 60-sq. ft. mine airways of the same absolute degree of roughness, the difference between absolute similarity and geometrical similarity had to be estimated. From rather scanty data the accompanying decrease in factor was estimated at about 25% and the tests on the Railroad Tunnel rock section exactly confirm this result if the assumption is made that the determined factors are not influenced by low-velocity flow. Test conditions did not permit determining the effect of the latter for the large railroad sections, but the results obtained indicate that it was either quite small or negligible. Its effect, if present, would be to increase the factor and to increase the percentage of decrease of factor over that determined. The uncertainty is thus on the safe side.

Effect of Low-Velocity Flow on Friction Factor.—Tests on the rock and timbered sections in the Moffat Water Tunnel show a rapidly increasing factor for velocities of less than about 250 ft. per min. Tests on the Railroad

Tunnel sections could only be made for a very limited range of low velocities, but both the determined value of the factors and their close agreement, for the small velocity range of 57 to 83 ft. per min. in the rock section, indicate that, for these large sections, the effect of low-velocity flow on the factor is either quite small or negligible.

Effect of Timber Spacing on Friction Factor.—The tests in the timbered sections of the Railroad Tunnel corroborated determinations made in the Water Tunnel which showed that, for the close spacing of 5 ft. or less, the friction factor decreased with the spacing of the sets. Inasmuch as laboratory tests tend to indicate that spacing timbers far apart would decrease the friction factor, there appears to be a critical spacing at which the air begins to form eddies between the timbers, and this spacing apparently bears a relation to the airway area. The results in the Railroad Tunnel can be attributed to the closeness of the timbers as compared with the area of the section, a ratio much smaller than that which exists in mine airways.

Although the purposes for which these tests were made did not justify further expenditure, the comparative data obtained indicate the possibilities of more extensive investigations and were found helpful in estimating the air friction of the completed Railroad Tunnel.

Ventilating Plant

A ventilating plant corresponding to these friction factor findings was built and is being used by the Denver and Salt Lake Railway Company for the steam operation of the railroad tunnel. A 500-h.p. fan with a capacity of 350 000 cu. ft. per min. and a stand-by 750-h.p. fan, with a capacity of 450 000 cu. ft. per min., are housed in a concrete building forming the East Portal.

They are connected to the tunnel by ducts with hinged steel dampers by means of which the direction of the air currents can be reversed, so that induced draft can be provided for westbound traffic and forced draft for eastbound. A vertically lifting canvas door on a steel frame closes the East Portal during periods of ventilation and is automatically raised when a train approaches. Air-current velocities vary from about 10 to 15 miles per hour.

Consulting Engineers.—The Board of Consulting Engineers retained by the Moffat Tunnel Commission was comprised of the late David W. Brunton, M. A. I. M. M. E., *Chairman,* J. Waldo Smith, Hon. M. Am. Soc. C. E., and L. D. Blauvelt and J. Vipond Davies, Members, Am. Soc. C. E.

This Board rendered valuable service during the construction of the project. When unexpected difficulties demanded utmost co-operation and resourcefulness it suggested, among other expedients, the use of the steel wall-plates. Its recommendation that the Water Tunnel be lined west of the apex is being gradually accomplished.

Economic Value of the Tunnel

The project involved $15 470 000 outstanding bonds. The first issue amounted to $6 720 000 at 5½%; the second, $2 500 000 at 5¼%; the third, $3 500 000 at 5¼%; and the fourth, $2 750 000 at 5 per cent. Real estate of

the Moffat Tunnel Improvement District, of which Denver represents 89%, is security for these bonds, and a total cost, including earnings of camps and interest during construction, of $18 000 000, in round figures, must be of pronounced economic value to justify this investment.

The purpose of the tunnel was threefold:

1.—To eliminate about 30 miles of 4% grade over Rollins Pass, lowering the maximum altitude from 11 660 ft. to 9 240 ft., which is below the snow areas, reducing the ruling grade to 2.0% and the maximum curvature to 12°, and eliminating 10 800° of curvature; all of which was important to the life of the Denver and Salt Lake Railroad.

2.—To place Denver on a through transcontinental rail route, saving 173 miles between Denver and Salt Lake City, by either building the 41-mile Dotsero Cut-Off, connecting the Denver and Salt Lake Railroad with the Denver and Rio Grande Western Railroad, or extending the Denver and Salt Lake Railroad from its present terminus, Craig, to Salt Lake City; and at the same time to assure adequate transportation facilities for a vast area in Northwestern Colorado rich in timber, coal, oil, oil shale, live stock, and agricultural and mining resources.

3.—To provide a conduit for the transportation of water from the Upper Colorado River on the western slope to the dry eastern slope around Denver for municipal and agricultural purposes.

Progress toward accomplishing these results may be noted:

(1) The physical railroad improvement has been accomplished.
(2) Negotiations for the construction and use of the proposed Dotsero Cut-Off are under way with both the Denver and Rio Grande Western and Denver and Salt Lake Railroads as probable users. Applications are before the Interstate Commerce Commission.
(3) The City of Denver has entered into a contract for the use of the Water Tunnel as a carrier for its Fraser River water. It is proposed under this contract that the City pay $1.00 per year rental, maintain the water bore, and eventually assume the cost of actual lining as part of its water-supply development program.

The indirect returns from these proposed uses of the tunnel should be increasingly great, obscuring the difficulties of construction and constituting another chapter in the story of the Moffat Tunnel.

APPENDIX I

MOFFAT TUNNEL—COST DATA

Itemized Cost of Rock Excavation in 8 by 8-Foot Tunnel and Headings.

(Average overbreak beyond 64 sq. ft. = 50%; cost based on yardage within 8 by 8-ft. pay line = 171 580 cu. yd.; these costs do not include plant cost, camp, or interest charges, or over-head beyond the superintendent.)

Item No.	Labor.	Cost per cubic yard.	Item No.	Supplies.	Cost per cubic yard.
(*a*)	Drilling and blasting	$1.564	(*a*)	Drill steel	$0.223
(*b*)	Hand-mucking (clean up)	0.454	(*b*)	Air hose	0.029
(*c*)	Machine mucking	1.051	(*c*)	Air pipe	0.205
(*d*)	Scaling	0.150	(*d*)	Water hose	0.014
(*e*)	Hauling in tunnel	0.955	(*e*)	Water pipe	0.075
(*f*)	Disposal	0.380	(*f*)	Ventilating pipe	0.129
(*g*)	Track-laying	0.660	(*g*)	Electric lights and wiring	0.392
(*h*)	Pipe lines	0.224	(*h*)	Trolley	0.037
(*i*)	Ventilation	0.221	(*i*)	Dynamite	1.497
(*j*)	Lights and wiring	0.259	(*j*)	Tamping rags	0.003
(*k*)	Power house	0.127	(*k*)	Fuse	0.020
(*l*)	Sharpening steel	0.336	(*l*)	Blasting wire	0.031
(*m*)	Repairs, mucking machines	0.153	(*m*)	Exploders	0.178
(*n*)	General repairs	0.162	(*n*)	Oil, grease, and waste	0.067
(*o*)	Drainage	0.357	(*o*)	Storage battery supplies and repairs	0.011
(*p*)	Superintendence	0.250	(*p*)	Locomotive repairs	0.077
(*q*)	Shifters	0.407	(*q*)	Car repairs	0.052
(*r*)	Drill repairs	0.059	(*r*)	Tool repairs	0.033
(*s*)	Car repairs	0.070	(*s*)	Track in tunnel and dump	0.300
(*t*)	Trolley	0.023	(*t*)	Drill parts	0.251
	Miscellaneous, including "jumbos", drayage, trimming, bonus, and special work	0.248	(*u*)	Mucker repairs	0.210
			(*v*)	Power	0.650
			(*w*)	Miscellaneous, including drayage	0.057
			(*x*)	Steel sharpening	0.105
			(*z*)	Tools, replacements	0.042
			(*a-b*)	Drainage	0.074
			(*a-a*)	Pumps	0.015
	Total labor cost per cubic yard	$8.110		Total supplies per cubic yard	$4.777

Total cost per cubic yard, labor and supplies $12.89

Total cost per cubic yard, labor and supplies, in solid rock $10.41

APPENDIX II

MOFFAT TUNNEL—COST DATA

Itemized Cost of Rock Excavation in Enlargement from 8 by 8-Foot to 16 by 24-Foot Standard, Single-Track Railroad Tunnel.

(Average overbreak = 18%; cost based on pay line yardage = 433 293 cu. yd.; these costs do not include plant cost, camp, or interest charges, or overhead beyond the superintendent.)

Item No.	Labor.	Cost per cubic yard.	Item No.	Supplies.	Cost per cubic yard.
(a)	Drilling and blasting	$0.996	(a)	Drill steel	$0.140
(b)	Hand-mucking (clean up)	0.684	(b)	Air hose	0.025
(c)	Machine mucking	0.481	(c)	Air pipe	0.039
(d)	Scaling	0.041	(d)	Water hose	0.009
(e)	Hauling in tunnel	0.597	(e)	Water pipe	0.001
(f)	Disposal	0.301	(f)	Ventilating pipe	0.001
(g)	Track in tunnel	0.115	(g)	Lights and wiring	0.093
(h)	Pipe lines	0.071	(h)	Trolley	0.018
(i)	Ventilation	0.001	(i)	Dynamite	0.433
(j)	Lights and wiring	0.081	(j)	Tamping rags	0.001
(k)	Power house	0.067	(k)	Fuse	0.035
(l)	Sharpening steel	0.165	(l)	Blasting wire	0.006
(m)	Air-shovel repairs	0.035	(m)	Exploders	0.045
(n)	General repairs	0.125	(n)	Oil, grease, and waste	0.043
(o)	Drainage	0.017	(o)	Storage battery repairs	0.002
(p)	Superintendence	0.099	(p)	Locomotive repairs	0.058
(q)	Shift bosses	0.304	(q)	Car repairs	0.029
(r)	Drill repairs	0.030	(r)	Tool repairs	0.024
(s)	Car repairs	0.049	(s)	Track repairs	0.098
(t)	Trolley	0.013	(t)	Drill parts and repairs	0.152
(u)	Trimming	0.097	(u)	Shovel repairs	0.050
	Miscellaneous (including "jumbos," drayage, special work, etc.)	0.206	(v)	Power	0.328
			(w)	Miscellaneous	0.034
			(x)	Steel sharpening	0.064
			(y)	Jumbo set	0.056
			(z)	Tool replacements	0.032
			(a-a)	Pump repairs	0.003
			(a-b)	Drainage	0.003
			(b-b)	Refuge bays	0.001
Total labor cost per cubic yard		$4.575	Total supplies per cubic yard		$1.823

Total cost per cubic yard, labor and supplies ..... $6.398 = $6.40

Total cost per cubic yard, labor and supplies, in solid rock $4.80

APPENDIX III

MOFFAT TUNNEL—COST DATA

Cost of Timber Lining

(In dollars per 1 000 ft. B. M. in place; for 1 000 ft. B. M. per linear foot of tunnel see attached typical timbering drawings; standard 5-segment, 12 by 12-in. set = 1 050 ft. B. M.; spark protection = 10-ft. wide canopy of 1 by 6-in. California Redwood nailed to permanent segmental arch timbers.)

Item.	Labor.	Material.	Total.
Oregon fir in railroad tunnel	57.11	52.67	109.78
Native Lodgepole pine in railroad tunnel	121.84	56.56	178.40
Timber in water tunnel	88.53	27.79	116.32
Cost per linear foot of tunnel	1.01	0.85	1.86

APPENDIX IV

MOFFAT TUNNEL—COST DATA

Average Cost of Steel Lining in 16 by 24-Foot Railroad Tunnel.

Item.	Labor.	Material.	Total.
Arch	$ 99.71	$252.88	$352.59
Pair of posts	151.19	353.15	504.34
Invert set	31.74	84.17	115.91
Complete set	$282.64	$690.20	$972.84
Cost per linear foot of tunnel sets on about 3-ft. centers	$112.34	$234.74	$347.08

Cost per linear foot of concrete lining (inverts, side-walls and arches), 8.5 cu. yd. at $31.74 = $270.00

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

FREQUENCY AND INTENSITY OF EXCESSIVE RAINFALLS AT BOSTON, MASSACHUSETTS

By Charles W. Sherman,* M. Am. Soc. C. E.

Synopsis

This paper presents the results of a study of the records of the recording rain gauge at Chestnut Hill Reservoir in Boston, Mass., covering a term of 50 years, and durations of heavy rain up to 72 hours. Curves of intensity of precipitation, corresponding to any period of time up to 72 hours, for various frequencies of occurrence are derived and discussed.

The form of curve showing the relation between the duration and the (average) intensity of precipitation is discussed, and the possibility of the application to other localities of the general formula herein derived, with the coefficients and exponents suitably modified, is suggested.

Intensity of Rainfall

The rate at which precipitation occurs during heavy storms has been recognized for many years as the most significant of the information required in the design of storm-water drains and combined sewers. At first the hourly rate, often taken as the presumable maximum for the locality, was commonly used, irrespective of the magnitude of the area to be drained. Later, it was recognized that the "time of concentration" for the particular area represented the period for which the rate of precipitation was desired; and it was noted that there was a relation between the length of the period and the corresponding intensity of precipitation.

For some time it was held that only periods of uniform intensity of precipitation were of significance in estimating storm-water run-off from rates of rainfall. In particular, the late Emil Kuichling, M. Am. Soc. C. E.,† emphasized the fact that if the rain fell at varying rates during the period of con-

Note.—Written discussion on this paper will be closed in **August, 1930**, *Proceedings*.

* Cons. Engr. (Metcalf & Eddy), Boston, Mass.

† "The Relation Between the Rainfall and the Discharge of Sewers in Populous Districts." *Transactions*, Am. Soc. C. E., Vol. XX (1889), p. 1.

centration, it was practically impossible to make trustworthy estimates of run-off from the rainfall record. If this criterion were rigidly applied, there would be comparatively few records of heavy rain of more than 20 or 30 min. duration which could be used; yet it is obvious that the effect of storms of variable intensity in producing run-off must usually be as great as, and sometimes greater than, that of a storm of uniform intensity and the same total precipitation in the particular period of time. Therefore, average intensities have been accepted as significant, and are utilized in the same way as though the precipitation fell at a uniform rate throughout the period under consideration; although it is recognized that the rate of rainfall was substantially uniform throughout the period in comparatively few cases where the duration exceeded 30 min.

Recording Rain Gauge at Chestnut Hill Reservoir

This gauge was established by the late Desmond FitzGerald, Past-President, Am. Soc. C. E., in 1878,* and there are now records from it covering a period of fifty years. It is now being maintained by the Metropolitan District Commission. A description of the gauge and its exposure is contained in a paper by Harrison P. Eddy, M. Am. Soc. C. E., entitled "Maximum Rates of Precipitation at Boston for Various Frequencies of Occurrence".†

Effect of Elevation of Gauge.—The collector of this gauge is on the roof of a building, about 25 ft. above the ground. An ordinary rain gauge is maintained at ground level, about 800 ft. from the present location of the recording gauge and about 450 ft. from the position which it occupied previous to 1898. In general, the total precipitation in any storm as shown by the elevated gauge was somewhat less than that collected in the ground gauge, although there were a few cases in which the former showed the larger amount.

The ground gauge has been assumed to be correct in all cases, and the records of the recording gauge have been adjusted by applying for each storm, a factor obtained by dividing the total collection, as shown by the ground gauge, by that indicated by the recording gauge. This factor has been applied to all periods of a storm, as there is no basis for modifying it. In value, it varied from 0.95 to 1.56; for the highest intensities, its value ranged from 1.03 to 1.16

Comparison with U. S. Weather Bureau Gauge.—In Mr. Eddy's paper referred to previously, the records of the gauge at the Chestnut Hill Reservoir, for the 38-year period then available, were analyzed and also those of the U. S. Weather Bureau gauge at Boston for 26 years. The distance between the two is about 5 miles. Although there were differences in the results obtained it was concluded that, on the whole, they were as nearly the same as could be expected; and as the records obtained at Chestnut Hill were adjusted by comparison with those of a ground gauge, while no such

* "Maximum Rates of Rainfall," by Desmond FitzGerald, Past-President, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. XXI (1889), p. 93.

† *Journal*, Boston Soc. of Civ. Engrs., 1920, Vol. VII, p. 47.

adjustment of the Weather Bureau gauge was possible, the former were thought to be the better.

In this paper, no comparison with the records of the U. S. Weather Bureau gauge is attempted.

Duration of Intense Rainfall.—In earlier analyses of records of intense rainfalls, comparatively little attention was paid to storms lasting more than 2 hours. In the cases in which longer periods were considered, it appears probable that but few of the significant storms were considered.

In view of the greater extent to which storm flow is now being estimated from rainfall, for periods of concentration as long as 2 or 3 hours, or even more, and of the recent application of this method in estimating flood flows of streams for which the period of concentration may be 3 days, or more, particular attention was paid in this analysis to heavy rains covering the longer periods. It was found that, in Mr. Eddy's paper, a number of storms which in reality lasted 3 hours or more at high rates were actually included in the study only for periods of 80 min., or less.

All storms which showed a precipitation of more than 1 in. in 24 hours, 2 in. in 48 hours, or 3 in. in total precipitation, were also studied in detail and rates of precipitation for longer periods up to 72 hours were computed and utilized in the present analysis.

"Extended Duration" Principle.—As a logical extension of the use of average rates of precipitation, all storms in which the total precipitation was sufficient to show significant average rates for periods longer than the actual duration of the rainfall were considered as though they had continued for the longer times. Thus, a storm which yielded a total precipitation of 1.50 in. in 50 min., had an average intensity of 1.80 in. per hour for that period, of 1.50 in. per hour for 60 min., and of 1.13 in. per hour for 80 min., all of which might be of significance in the analysis.

In tabulating the 50 highest intensities for each duration from 5 to 180 min., only 10 of the 650 figures were supplied in this way. In the case of long-time storms, however, a considerable number were tabulated for longer periods than the actual durations of the storms.

Analysis of Records.—The quantities of precipitation, adjusted to agree with the ground gauge, and the corresponding intensities for periods of 5, 10, 15, 20, 25, 30, 45, 60, 80, 100, 120, 150, and 180 min., were tabulated for the significant storms, so as to obtain the fifty greatest intensities for each of these periods, utilizing the "extended duration" principle wherever applicable. The records of long storms were analyzed by hours, to ascertain the greatest intensities for periods of 4, 5, 6, 8, 10, 12, 15, 18, 21, 24, 30, 36, 42, 48, 54, 60, 66, and 72 hours, so as to obtain at least the highest ten intensities for each of these periods.

FREQUENCY

If the 50-year record were a fair representation of the average 50-year period at Boston, then the greatest intensity for each of the periods should be that which is likely to recur once in 50 years, or that which is likely to have a frequency of 50 years. The points of second magnitude represent

intensities which might be expected to be equalled or exceeded twice in 50 years, or to have a frequency of 25 years; and the points of the fifth magnitude would have a frequency of 10 years.

It is reasonable to suppose, however, that some of the maximum points may be high enough so that similar intensities will not recur for 100, or even 500 years; the 500-year maximum must occur within some 50-year period, and it may have been in that studied. It is probable also that some of the maximum points may be lower than they would be for an average frequency of 50 years, if the record had covered a longer term, such as 200 years. That is, the curve showing the relation of intensity to duration for a frequency of 50 years cannot be very satisfactorily determined from the maximum points of a 50-year record. The 25-year curve is much more definitely fixed, because the record covers two full periods of 25 years; and curves of greater frequency are still more definitely determined.

Form of Intensity-Duration Curve

Intensity-duration curves for the several frequencies which might be expected to be fairly reliable, namely, those occurring three times in 50 years, and oftener, showed that the curves for all frequencies were of the same general form, although there were several places where the smooth curves were at some distance from the plotted points. The similarity in form of curve suggested that a single form of equation, with suitable modification of the constants, might be applicable to intensity-duration curves for all frequencies. Plotting on logarithmic cross-section paper with intensities as ordinates and durations as abscissas resulted in nearly parallel lines of slight curvature, which were convex upward. Then, assume an equation of the form:

$$i = \frac{K}{(t+b)^d} \quad \ldots\ldots (1)$$

in which, i represents intensity of precipitation, in inches per hour; t, the period of time, in minutes; and K, b, and d are constants; it was found by trial that a value of $b = 7.0$ was reasonably applicable to the points of each set (that is, for a particular frequency). Values of i were then plotted as ordinates against $t + 7$ as abscissas, and it was found that all the points of each magnitude, from the third to the fiftieth, would lie very nearly on a straight line, while those for the first and second magnitudes were reasonably distributed about such lines. They were very nearly parallel; and if d were taken as 0.7 in each case, the resulting parallel lines were satisfactory as representing the whole series of observations. The general formula for relation between intensity of precipitation and length of period is, therefore,

$$i = \frac{K}{(t+7)^{0.7}} \quad \ldots\ldots (2)$$

Value of Coefficient (K).—Values of K corresponding to the third, fifth, tenth, twenty-fifth, and fiftieth magnitudes have been taken from a diagram

upon which the lines representing Equation (2) were drawn, with the following results:

Frequency, in years.	Values of K.
1	16.0
2	20.0
5	24.8
10	30.5
16⅔	34.6

Values for frequencies of 25 and 50 years were not obtained by drawing intensity-duration curves, since the points departed so much from any straight lines (on logarithmic paper) that the positions of the most probable lines could not be fixed with satisfaction, at least on the basis of the study of precipitations of durations up to 180 min. It appears, however, that the relation between values of K as listed herein can be expressed satisfactorily by the equation:

$$K = 16\,F^{0.27} \qquad (3)$$

in which, F represents frequency, in years. The resulting values of K, compared with those taken from the diagram, are shown in Table 1.

TABLE 1.—VALUES OF COEFFICIENT AND RATIOS OF INTENSITY CORRESPONDING TO VARIOUS FREQUENCIES OF INTENSE RAINFALL.

Frequency, in years.	Values of K.		Ratio of Intensity to That for:	
	Computed.	From diagram.	1-year frequency.	10-year frequency.
1	16.0	16.0	1.00	0.54
2	19.3	20.0	1.21	0.65
5	24.7	24.8	1.54	0.83
10	29.8	30.5	1.86	1.00
15	33.2	....	2.08	1.11
16⅔	34.0	34.6	....	....
25	38.1	....	2.38	1.28
50	45.9	....	2.87	1.54
100	55.5	....	3.47	1.86

There is doubt as to the propriety of applying the computation to a frequency of 100 years, but it is interesting to note the results obtained and compare them with the greatest figures obtained from the records; and they are probably of significance as giving some indication of what may occur sometime in the way of phenomenal precipitation.

Relative Intensities for Various Frequencies.—The relationship shown by Equation (2) indicates that the intensity to be expected for any frequency of recurrence bears a direct relation to that for any other frequency; some of the ratios are arranged for comparison in Table 1.

COMPARISON BETWEEN COMPUTED AND OBSERVED VALUES

Fig. 1 shows graphically the relation between the length of period (duration) and the intensity obtained from the records of the storms of first, second, third, fifth, and tenth magnitudes for all periods up to 72 hours, and of the twenty-fifth and fiftieth magnitudes for periods up to 3 hours. These

magnitudes are indicated by numerals beside the observed points. Fig. 1 also shows the curves derived for frequencies of 100, 50, 25, 16⅔, 10, 5, 2, and 1 years. The use of logarithmic cross-section paper makes it possible to bring all this information within the limits of a single chart.

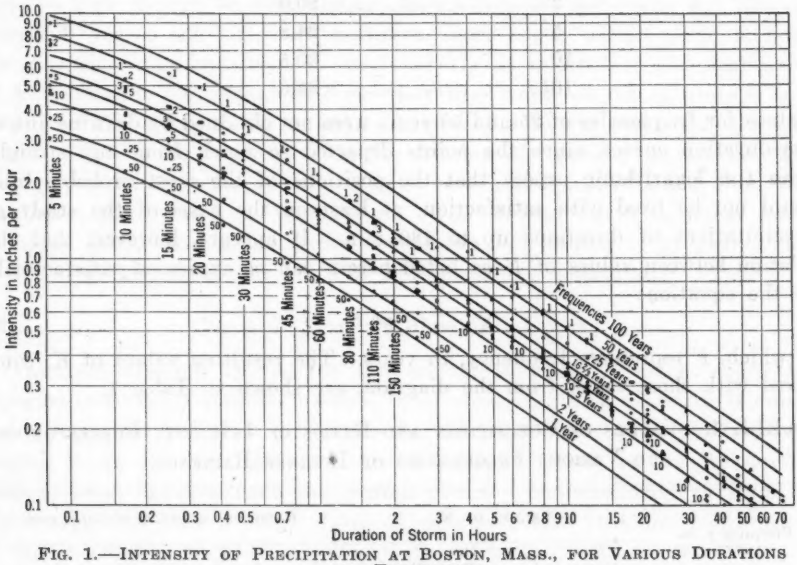


FIG. 1.—INTENSITY OF PRECIPITATION AT BOSTON, MASS., FOR VARIOUS DURATIONS AND FREQUENCIES.

The comparisons are believed to justify the conclusions that Equations (2) and (3) do represent satisfactorily the conditions existing at Boston for frequencies up to 50 years and durations up to 72 hours. There does not seem to be any record of a really excessive storm lasting more than 3 days, and it seems likely that such an occurrence, like total rainfalls in excess of 10 in. in a single storm, will be of great rarity.

Comparison with Previous Studies.—Some of the most significant of the earlier records observed at this station were the basis of another paper by the writer,* but the question of frequency was not discussed. Few if any engineers at that time had definite ideas on the relative frequency of rainfalls of high intensity, and records of rates of precipitation were insufficient to warrant any positive conclusion on that matter. A general form of equation was not developed, although two curves were suggested, as follows:

For maximum intensities,†

$$i = \frac{38.64}{t^{0.687}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

and for use as a basis for design,‡

$$i = \frac{25.12}{t^{0.687}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

* "Maximum Rates of Rainfall at Boston," by Charles W. Sherman, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LIV (1905), p. 173.

† *Loc. cit.*, p. 178.

‡ *Loc. cit.*, p. 179.

It is interesting to note that the curves represented by Equations (4) and (5) compare reasonably well with those derived herein for 50-year and 10-year frequencies, respectively.

Application of Formulas to Other Localities

It seems logical to assume that formulas of the same kind may be applicable in other localities. The writer has not found opportunity to make a similar analysis of other rainfall records; but a study made in 1920* resulted in the conclusion that the curve of rainfall intensities for the 15-year frequency at Boston, as given by Mr. Eddy† was a good one for a 5-year frequency at Springfield, Mass.; and that, in general, rainfall curves for Springfield were nearly like those which had been published for New York City. This indicates the possibility that suitable variations in the values of K may be all that is necessary to make the formula herein derived applicable to a considerable area, such as New England and New York. In other localities, variations in one or both of the constants, b and d, in Equation (1) probably will be required.

Occurrence of Excessive Rates During Storms

In fixing run-off coefficients for use in computing storm flow from rainfall, some engineers use the so-called "zone principle" according to which the area is assumed to be divided into zones on which the rain has been falling for different lengths of time. The assumption is usually made that the rate of precipitation is greatest at the beginning of the storm, and, therefore, that smaller coefficients of run-off can be made with higher, than with lower, rates of precipitation. Comparatively little definite information as to the correctness of this assumption is available, and, therefore, this study was extended to determine the significant data as to the time, with reference to the beginning of the storm, when the excessive precipitation actually occurred.

The records of the ten highest intensities, having durations from 15 to 180 min., were examined to ascertain the quantity of rain falling in a period of equal duration and immediately preceding that in which the intense rainfall occurred. The results are shown in Table 2. They indicate that the assumption of a substantially dry ground surface at the beginning of the intense precipitation is reasonably correct; although there have been cases of heavy rains (not those of greatest average intensity), in which the quantity falling before the intense precipitation began was considerable. The latter condition might result in rates of run-off substantially as high as those from more intense rain falling on practically dry ground.

Storms of Long Duration

It was noted previously that the study of long-duration storms was undertaken because it was believed that any formula representing the relation between intensity of precipitation and duration of precipitation should give

* "Excessive Rainfall Frequencies at Springfield, Mass.," by Charles W. Sherman, M. Am. Soc. C. E., *Engineering News-Record*, September 2, 1920, Vol. 85, p. 445.

† *Journal*, Boston Soc. of Civ. Engrs. (1920), Vol. VII, p. 47.

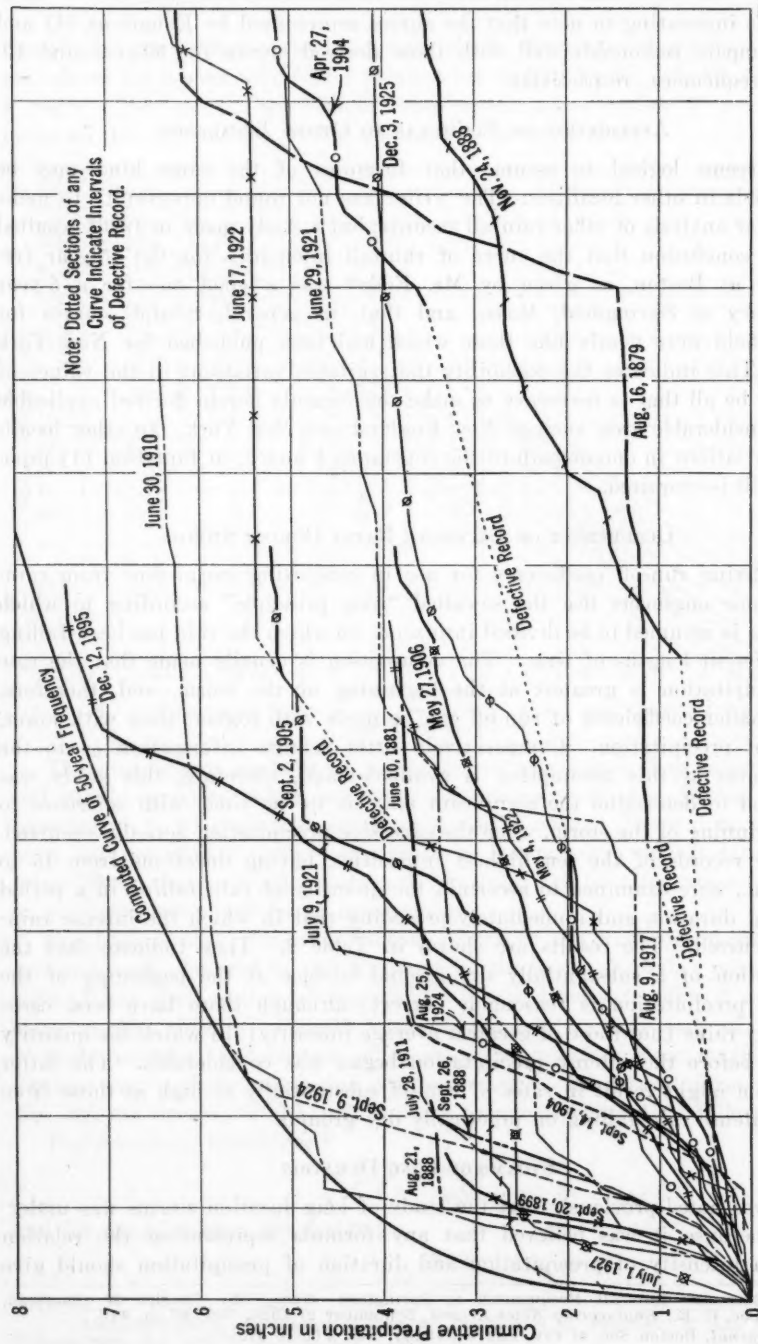


FIG. 2.—STORMS OF LONG DURATION, CHESTNUT HILL, BOSTON, MASS., 1879 TO 1928, INCLUSIVE.

TABLE 2.—QUANTITIES OF RAINFALL IN PERIODS OF EQUAL DURATION IMMEDIATELY PRECEDING THAT IN WHICH THE INTENSE RAINFALL OCCURRED.

Order of magnitude.	Date.	Intensity, in inches per hour.	Total amount before maximum period from beginning of storm, in inches.	Amount in equal period preceding the maximum period, in inches.	Amount during period of maximum rate, in inches.	Average intensity during equal period preceding maximum period, in inches per hour.
			15-MINUTE INTENSITIES			
1	Aug. 22, 1899	5.67	0.26	0.26	1.42	1.04
2	July 9, 1921	4.16	0.30	0.24	1.04	0.96
3	July 20, 1880	4.14	0.38	0.29	1.035	1.16
4	July 21, 1894	3.50	0.20	0.20	0.88	0.80
5	Aug. 25, 1901	3.44	0.47	0.18	0.86	0.72
6	July 18, 1884	3.42	0.35	0.02	0.86	0.08
7	Aug. 6, 1893	3.36	0.00	0.00	0.84	0.00
8	Sept. 25, 1911	3.04	0.27	0.27	0.76	1.08
9	July 10, 1921	3.00	1.02	0.18	0.75	0.72
10	Aug. 12, 1892	2.78	0.01	0.01	0.69	0.04
			30-MINUTE INTENSITIES			
1	Aug. 22, 1899	3.51	0.00	0.00	1.76	0.00
2	July 9, 1921	3.18	0.30	0.30	1.59	0.60
3	July 20, 1880	2.71	0.08	0.07	1.36	0.14
4	Sept. 25, 1911	2.63	0.14	0.14	1.32	0.28
5	Aug. 12, 1892	2.50	0.01	0.01	1.25	0.02
6	Aug. 25, 1901	2.31	0.33	0.12	1.155	0.24
7	July 21, 1894	2.22	0.00	0.00	1.11	0.00
8	Aug. 10, 1917	2.05	0.65	0.25	1.03	0.50
9	July 10, 1921	1.99	0.84	0.06	0.99	0.12
10	Aug. 21, 1888	1.96	1.75	0.55	0.98	1.10
			45-MINUTE INTENSITIES			
1	July 9, 1921	2.78	0.12	0.12	2.09	0.16
2	Aug. 22, 1899	2.49	0.00	0.00	1.87	0.00
3	Aug. 25, 1901	2.03	0.32	0.21	1.52	0.28
4	July 20, 1880	1.93	0.01	0.01	1.45	0.01
5	Sept. 25, 1911	1.93	0.01	0.01	1.45	0.01
6	Aug. 12, 1892	1.84	0.01	0.01	1.38	0.01
7	Aug. 21, 1888	1.72	1.75	0.61	1.29	0.81
8	June 30, 1915	1.62	0.63	0.34	1.22	0.45
9	Sept. 7, 1891	1.61	0.42	0.10	1.21	0.13
10	Aug. 10, 1917	1.57	0.50	0.26	1.18	0.35
			60-MINUTE INTENSITIES			
1	July 9, 1921	2.20	0.06	0.06	2.20	0.06
2	Aug. 22, 1929	2.04	0.17	0.17	2.04	0.17
3	Aug. 25, 1901	1.80	0.24	0.02	1.80	0.02
4	Aug. 21, 1888	1.58	1.55	0.83	1.58	0.83
5	Aug. 12, 1892	1.57	0.01	0.01	1.57	0.01
6	June 30, 1915	1.57	0.63	0.40	1.57	0.40
7	July 20, 1880	1.50	0.01	0.01	1.50	0.01
8	Sept. 25, 1911	1.45	0.01	0.01	1.45	0.01
9	Sept. 7, 1891	1.39	0.39	0.13	1.39	0.13
10	June 19, 1884	1.33	0.09	0.09	1.33	0.09

FIG. 2.—STORMS OF LONG DURATION, CHESTNUT HILL, BOSTON, MASS., 1879 TO 1928, INCLUSIVE.

TABLE 2.—(*Continued.*)

Order of magnitude.	Date.	Intensity, in inches per hour.	Total amount before maximum period from beginning of storm, in inches.	Amount in equal period preceding the maximum period, in inches.	Amount during period of maximum rate, in inches.	Average intensity during equal period preceding maximum period, in inches per hour.
			120-Minute Intensities			
1	June 30, 1915	1.28	0.37	0.37	2.56	0.19
2	Aug. 21, 1888	1.22	0.93	0.85	2.44	0.43
3	July 9, 1921	1.18	0.00	0.00	2.36	0.00
4	Aug. 22, 1899	1.17	0.00	0.00	2.33	0.00
5	Sept. 20, 1899	1.07	0.63	0.55	2.14	0.28
6	Aug. 25, 1901	0.94	0.21	0.11	1.88	0.06
7	July 20, 1880	0.89	0.01	0.01	1.78	0.01
8	Aug. 10, 1917	0.88	0.34	0.15	1.76	0.08
9	July 9, 1921	0.86	0.24	0.01	1.72	0.01
10	Aug. 12, 1892	0.85	0.00	0.00	1.70	0.00
			180-Minute Intensities			
1	June 30, 1915	1.12	0.63	0.63	3.37	0.21
2	Aug. 21, 1888	1.00	0.36	0.36	3.01	0.12
3	Sept. 20, 1899	0.83	0.43	0.43	2.49	0.14
4	July 9, 1921	0.82	0.00	0.00	2.46	0.00
5	Aug. 22, 1899	0.78	0.00	0 00	2.33	0.00
6	Sept. 10, 1924	0.76	3.09	2.11	2.28	0.70
7	Aug. 1, 1927	0.68	trace	trace	2.04	0.00
8	Aug. 16, 1879	0.67	1.57	0.00	2.01	0.00
9	July 9, 1921	0.63	0.23	0.00	1.89	0.00
10	Aug. 10, 1917	0.63	0.25	0.25	1.89	0.08

as reasonable results for periods of 1 day, or more, as for 1 hour or 2 hours, and also because of the increasing use of rainfall records in estimating probable flood flows of streams. In order to visualize the way in which the rain falls in such storms, the records of cumulative amounts in the twenty-two storms of greatest significance which occurred at Chestnut Hill Reservoir in the 50-year period, 1879-1928, have been plotted in Fig. 2. The curves show the computed maxima to be equalled or exceeded at average intervals of 50 years, according to Equation (2).

Acknowledgments

The writer is indebted to William E. Foss, Director and Chief Engineer, Water Division, Boston Metropolitan District, for permission to make use of the records contained in this paper, and to John W. Raymond, Assistant Engineer, Metcalf and Eddy, Boston, for assistance in the preparation of the paper.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

THE SHANNON POWER DEVELOPMENT IN THE IRISH FREE STATE

By A. R. C. Markl,* Jun. Am. Soc. C. E.

Synopsis

This paper aims to present a broad outline of the civil engineering part of the construction of the Shannon Power Development in the Irish Free State. The work was done by German contractors and the construction methods used are representative of the best European practice.

Inasmuch as the manner of dealing with the concrete and rock work did not differ appreciably from American practice under similar circumstances, this part of the description will be restricted to a brief recital of the features peculiar to this particular construction.

The equipment and methods used in building a large waterway, such as the head-race of the power plant, however, present features which are not only fundamentally different from American practice, but which are also without precedent in Europe. With the enormous task of building levees on the Mississippi looming before American engineers, it is thought that a detailed account of the methods used in excavation and building embankments should prove of particular interest.

General Description of the Development

Southern Ireland with its 25 000 sq. miles and 3 000 000 inhabitants has very little industry. It uses about 45 000 000 kw-hr. per year; that is, only 15 kw-hr. per person per year as against 650 kw-hr. in the United States. This power is supplied by small local stations at prices ranging as high as 20 cents per kw-hr. When the Irish Free State achieved its independence, the Government recognized as one of its foremost tasks the development of the country's latent power resources, with a view to infusing new vigor into its economic life. As early as 1925 the Shannon Electricity Act was passed by the Oireachtas, authorizing the construction of a water power plant on the River

Note.—Written discussion on this paper will be closed in **August, 1930,** *Proceedings.*

* Civ. Engr., Stevens & Wood, Jackson, Mich.

Shannon. It is Government-owned, and will supply a minimum of 150 000 000 kw-hr. in 1930, and a maximum of 500 000 000 kw-hr. in a later, more complete, development. The cost of the first part of the work, exclusive of the transmissions and transformer stations, was $16 500 000. Basing estimates on amortization in 40 years and 5.5% interest, and including operating expenses and over-head, the cost per unit at the power house will be 0.80 cent for the first part of the development, and 0.45 cent for the full development. The corresponding cost at the 38-kv. bus-bars at Dublin, Cork, and Limerick, which cities are interconnected by 110-kv. loops, is estimated at a little more than 1 cent per kw-hr.

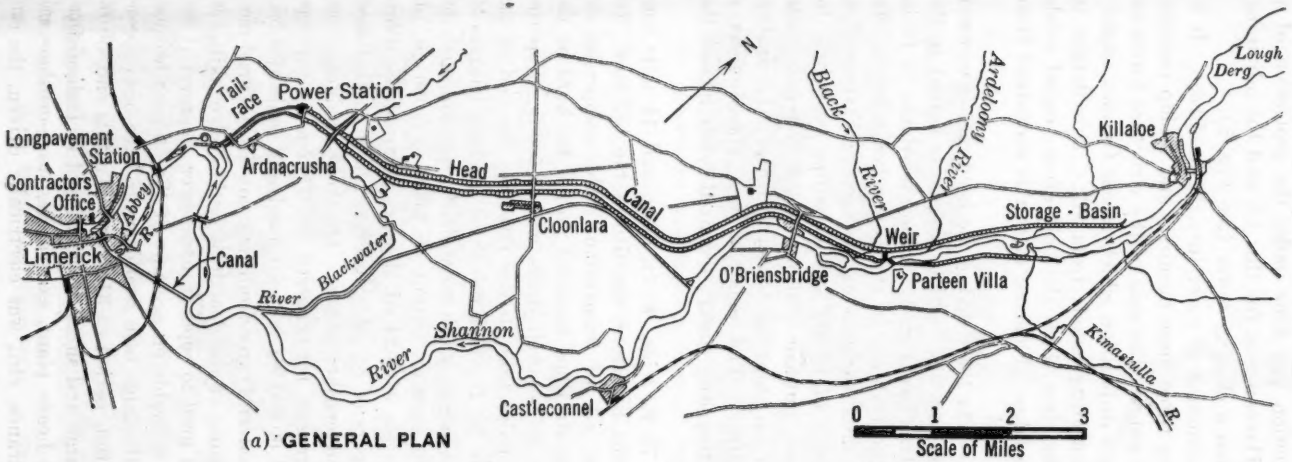
The conditions for the construction of a large water power plant near Limerick may be considered as favorable. The River Shannon is about 200 miles long, and on its course through the center of the island drains about one-sixth its total area, or 4 000 sq. miles, with a mean annual rainfall of 37 in. The average flow in its lower reaches is 8 500 sec-ft., the highest floods carry 32 500 sec-ft., and the lowest run-off observed was 900 sec-ft.; the ratio of the two latter values is 36 : 1, which compares very favorably with most European rivers. The upper course of the Shannon has very little slope, since the river flows through three extensive lakes, Lough Allen, Lough Ree, and Lough Derg, with a total surface area of more than 100 sq. miles, which serve as natural storage basins. The greatest drop is concentrated between the last-named lake and the tidal estuary of the river, and here it is possible to utilize a head of 100 ft. in a stretch of 17 miles.

Principal Construction Items

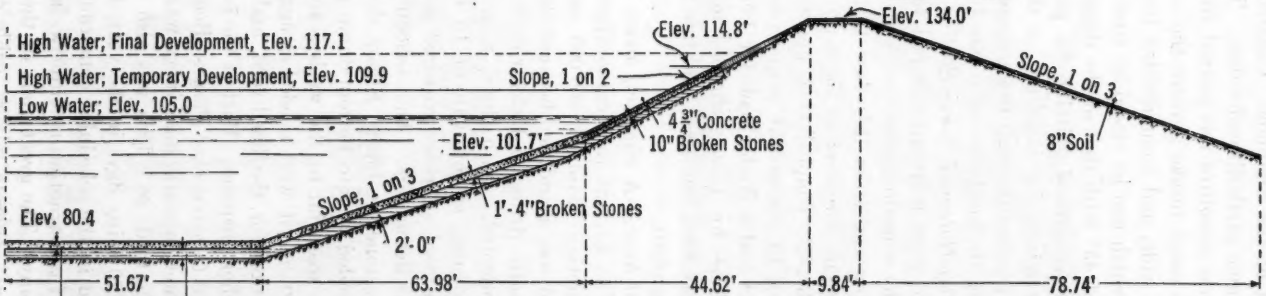
The actual work which had to be undertaken may be subdivided into the following main sections: (1) Raising the banks of the lower lake (Lough Derg) to increase its storage capacity; (2) construction of a diversion dam and canal head-gates; (3) head-race construction; (4) construction of the water-gate, the power station, and ancillary, or secondary, works; (5) tail-race excavation.

Storage Capacity Increase.—Despite their large area, the existing natural lakes were insufficient to provide the storage space required for balancing the water supply between periods of flood and drought. Low embankments had to be thrown up along the sides of the lower lake, and parts of the Upper Shannon had to be regulated. Incidentally, these measures will benefit agriculture by protecting large areas from inundation, and shipping by providing a deeper and straighter navigable channel. This work was done mainly by means of small bucket excavators.

Diversion Dam and Head-Gates.—Six miles below Lough Derg (Fig. 1(*a*)), a short distance down stream from a geological fault crossing the Shannon at right angles, a dam which raises the water 25 ft. was built on the crest of the hard red sandstone stratum sloping up against the river at this point. This dam is of the ogee overflow type, and has an over-all width of 380 ft.; the form producing favorable flow and minimum toe erosion was determined by model tests in a scale of 1 : 50 made at the Hydraulic Laboratory of the Technical University of Berlin. Six steel gates, two of which serve as scour-



(a) GENERAL PLAN



(b) TYPICAL HALF-SECTION OF THE HEAD-RACE OF THE POWER HOUSE

FIG. 1.—PLAN AND SECTION OF THE SHANNON POWER DEVELOPMENT.

ing sluices, rest on the weir crest, and allow of discharging floods into the old bed of the Shannon. Construction was done under the protection of a steel-frame earth-fill coffer-dam. The concrete for this job and the adjoining head-gate structure was poured from a chuting tower (see Fig. 2).

The canal intake governs the admission of water into the head-race. It is 320 ft. wide, and is subdivided into three main openings and a ship passage, all of which can be closed by steel gates. The axis of the structure forms an angle of 45° with that of the dam, a disposition which recent German experiences have proved suitable for protecting canals from silt. This danger is already minimized inasmuch as the river sheds the bulk of suspended solids on its passage through the storage lakes. Floating matter is restrained from entering the head-race by a panel wall.

Water-Gate and Power Station.—The lower end of the head-race is widened so as to form a trumpet-shaped forebay, and further passage is barred to the water by a medium-size concrete dam (see Fig. 3). Short penstocks 20 ft. in diameter lead from this dam to the turbines in the power house below. In the present development there are three turbines of the vertical Francis type, each directly coupled to a generator of 30 000 kv-a. nominal capacity. In addition to the actual power plant, outdoor station, etc., a concrete waste channel and a double lock for 150-ton ships have been provided. Excavation was done by power shovels mainly. The concrete was placed from a cableway with one fixed and one traveling tower; the fixed tower housed the mixing plant.

Tail-Race.—A channel, about 75 ft. wide, 40 ft. deep, and 1¼ mile long (see Fig. 5), cut into carboniferous limestone constitutes the tail-race. Its construction involved 800 000 cu. yd. of excavation. The compressed-air method was abandoned because the stratified material proved too hard on the pneumatic drills, and electrically driven well drills with 1 500-lb. drop-bars were adopted. With these, 8-in. holes were driven down to the bed of the future canal at intervals of 15 to 20 ft., and the three or four charges of ammoniated saltpeter required per canal width were fired simultaneously by electric detonators. This schedule gave an output of 4 000 to 5 000 cu. yd. of limestone per blast. Steam shovels with 2½-yd. dippers removed the débris and loaded it into bottom-dump cars of 8½ yd. capacity, which carried most of the material to the washer and crusher plants. Of their product, about 400 000 cu. yd. were needed as aggregate for concrete, 500 000 cu. yd. for slope protection for the head-race, and 60 000 cu. yd. for road and railway work, and other purposes. Part of the material came from rock cuts in the head-race.

The Construction Plant.—Because of the dearth of native labor skilled in construction work, machinery was used to replace man-power wherever it was possible to do so. This was also desirable on account of the climate because, with 270 rainy days in the year, the huge task would never have been accomplished in the scheduled construction period of 3½ years. To this end, 30 000 tons of construction machinery, large and small, were shipped to Ireland from Germany. The great mass of the heavy plant, excavators, stone-crushers and washers, concrete mixers, cable cranes, etc., was electrically driven, the current being supplied by a temporary power station with nine Diesel engines of

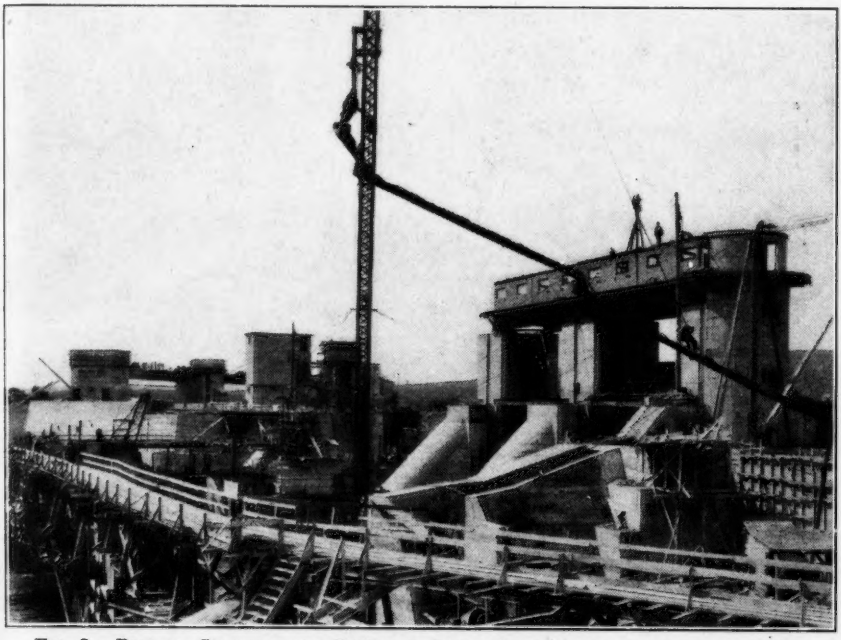


FIG. 2.—POURING CONCRETE FOR DIVERSION DAM. HEAD-GATES TO BE SEEN IN REAR.

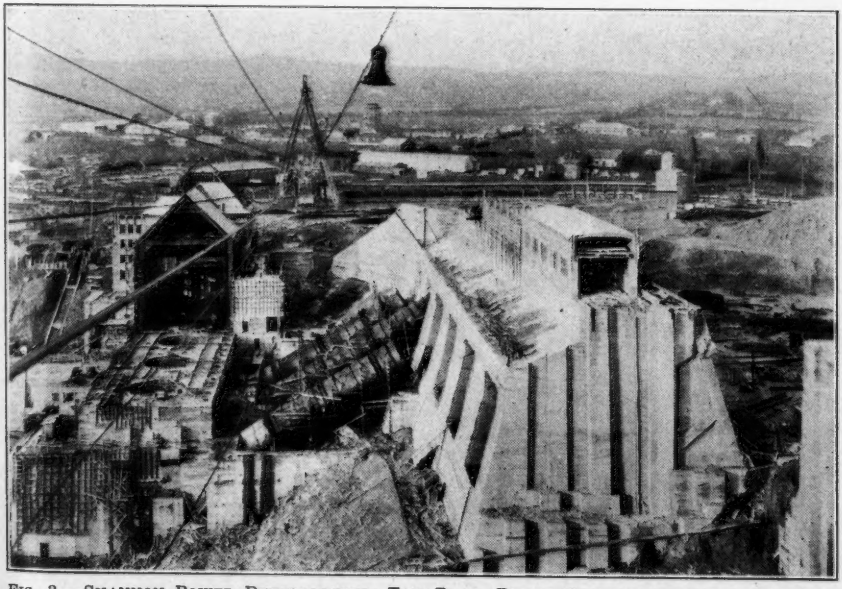


FIG. 3.—SHANNON POWER DEVELOPMENT: TAIL-RACE, POWER STATION, PENSTOCKS, WATER-GATE, FOREBAY OF HEAD-RACE. IN REAR, FIXED TOWER OF CABLEWAY.

4 500 h.p. total capacity. The absence of any local steel industry forced the contractor to build a repair shop employing 300 workmen day and night, and to order machinery with interchangeable parts so as to keep within reasonable limits the number of spare parts to be stored.

Canal Construction Methods

Design of Head-Race.—While the power station, the tail-race, and all work up stream from the dam, were developed for the anticipated needs of the first ten years of operation only, with due forethought for extension in the future, the head-race was completed at once for the so-called full development, or the final stage of the work. A policy of restricting construction of the head-race to immediate requirements would not have been justifiable economically, inasmuch as present gains would have been more than offset by future excess expenditure for enlarging the waterway.

The head-race, therefore, was built for a capacity of 19 500 sec-ft. Since it was also to serve as a navigation channel, the flow velocity was limited to a maximum of 5 ft. per sec., and the cross-section planned accordingly.

A typical half-section is shown in Fig. 1 (*b*). The head-race has the notable width of 320 ft. between the crowns of the banks, and a depth of as much as 37 ft. in the final development. Only a short length of the canal is entirely in "cut", most of it being constructed partly in cut and partly as fill. In places where it crosses deep gullies, the fill attains a height of 60 ft., with a span of 700 ft. between the outer toes.

With a view to providing good flow conditions, the inner slopes were banked to an inclination of 1 on 3 near the bottom and 1 on 2 at the top. The outer slope of 1 on 3 assured adequate stability for embankments as high as 25 ft. Where the fill exceeded this height, berms were provided, and the slopes were flattened to 1 on 4 and 1 on 5.

Since the total earth excavation was approximately 10 000 000 cu. yd., and similar quantities had to be placed in the enbankments, proper selection of equipment and good disposition thereof became vital problems.

Preparation of Site.—The greater part of Ireland is pasture land, each individual plot being fenced in by stone walls or hedges. These obstacles had to be removed first. Walls and tree stumps were blasted out, and small steam shovels on caterpillars did excellent service in removing stone heaps, underbrush, and hedges.

The area to be occupied by the future canal and embankments was then stripped of surface soil by means of multiple-bucket excavators on caterpillars with 35-ft. conveyor booms, and the earth dumped aside for re-soiling the embankment slopes. Usually two machines worked in series, one of them stripping, and the other rehandling, the material.

Excavation.—After these preparations, the actual excavation could be undertaken. The question arose, as to whether to use power shovels or multiple-bucket excavators. Conditions were such that, at first, neither equipment had any apparent advantage over the other, and only after long and careful study of their respective merits was the decision given in favor of the latter.

To be sure, its ruggedness, mobility, and general usefulness make the power shovel a very valuable tool, especially where excavation is irregular in plan or section and high resistances are encountered; and, therefore, as a matter of course, shovels of various capacities were adopted for foundations and rock work, at the power house and dam sites, as well as in the tail-race.

In the head-race, on the other hand, a shallow trough had to be formed true to design dimensions, and, if possible, in one single operation. The multiple-bucket excavator was found to be the only machine capable of excavating the canal bed and slopes, including breaks in the slopes, without rehandling material or much finishing (see Fig. 4). It raises the excavated material to surface level, and thus eliminates the expensive track work and up-hill haul required in the case of a power shovel operating in the cut, an advantage which is further accentuated where clayey water-bearing soil is encountered. However, it must be conceded that the tenacious marley clay strewn with boulders, which formed the bulk of the excavation on the Shannon work, was no more favorable to the multiple-bucket excavators than to the power shovels. In fact, this material marks the limit of usefulness of this type of excavator; but, even so, with only 30 to 40% of its theoretical capacity, it proved to be the economical superior of the power shovel for this kind of work.

The following are the principal data for one of the excavators used: With a bucket capacity of $\frac{1}{3}$ cu. yd., and 22 to 28 buckets discharged per min., the excavator capacity is 500 cu. yd. per hour theoretically. Under favorable soil conditions, 350 cu. yd. should have been an easy average, but actually no more than 200 cu. yd. were obtained on this job. The length of the bucket ladder is 90 ft., and the maximum depth of excavation is 58 ft. By changing the ladder rigging, the machine can also be made to scrape material down from elevations higher than the track, in which case a height of 46 ft. is the limit. The machine travels at a speed of 16 ft. per min. on the track, that is, in a direction transverse to the bucket chain, and is capable of climbing grades as steep as $2\frac{1}{2}$ per cent. Its service weight is 220 tons.

Portable transformer wagons step the 17 300-volt current supplied by the auxiliary power station down to 3 000, which is the voltage of the main drive, and a transformer inside the excavator steps it down farther to 380 and 220 volts, the tensions at which the advancing drive, the winch drive, and the compressor motor work. The machine requires 150 h.p.

The following were the reasons for adopting electric drive in place of steam for this and other equipment. An electric multiple-bucket excavator uses 0.4 to 0.6 kw-hr. per cu. yd., while a steam-driven machine of the same capacity consumes 3 to $3\frac{1}{2}$ lb. of coal. Considering the necessary haulage in the case of coal, and including the amortization of the auxiliary power plant and transmissions in the price of current, costs per unit excavated and loaded are about balanced; but electric drive has the advantages of higher over-load capacity, instantaneous readiness, and elimination of delays for firing up; it needs no stoker and, therefore, can be operated by one man; and it requires no water stations and less lubricants. Furthermore, it is cleaner and easier

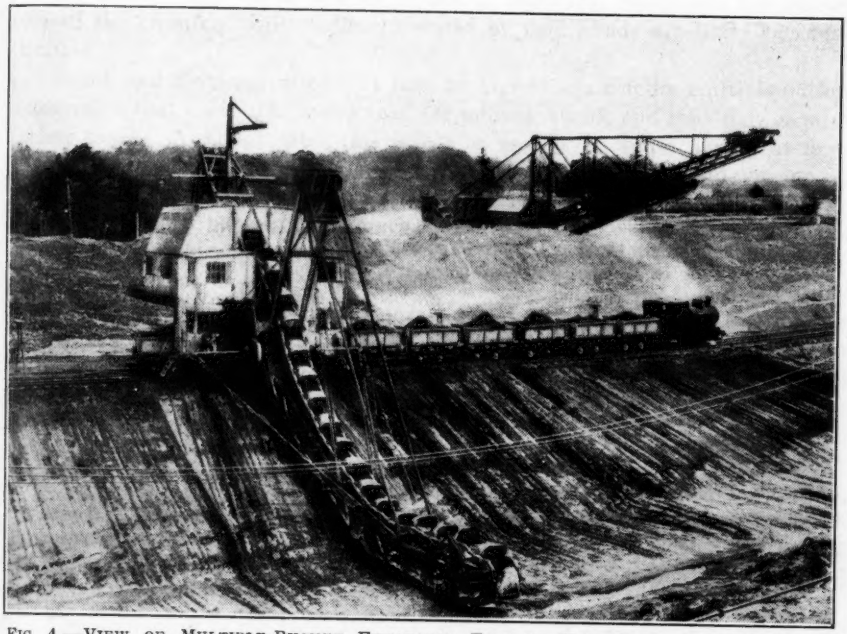


FIG. 4.—VIEW OF MULTIPLE-BUCKET EXCAVATOR FORMING HEAD-RACE, SHANNON POWER DEVELOPMENT. IN REAR, BANK-BUILDING MACHINE.

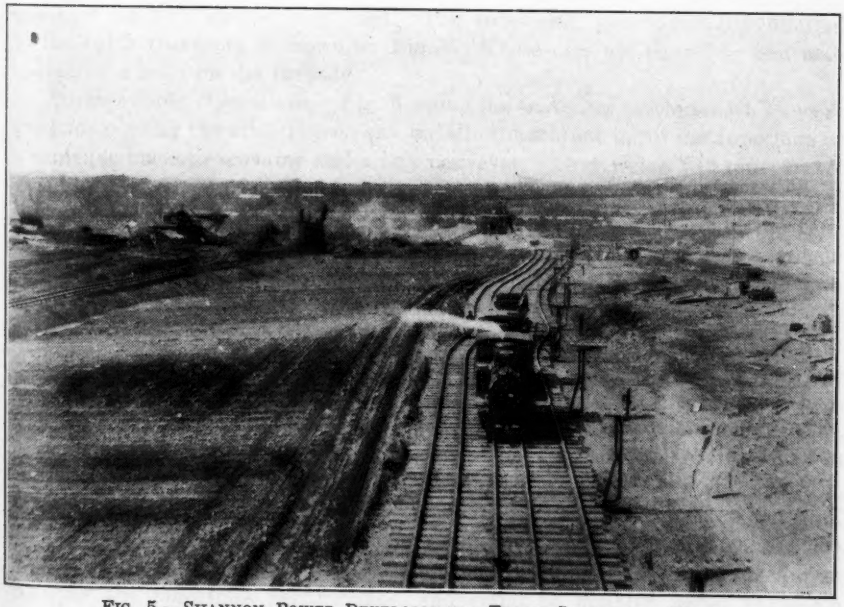


FIG. 5.—SHANNON POWER DEVELOPMENT: TRACK SHIFTER AT WORK.

to supervise; break-downs due to failure of electrical equipment are almost out of the question, while boilers operated by field hands are likely to cause trouble.

Track and Transportation.—It may be argued against the multiple-bucket excavators that their size is awkward for moving about, and that they require heavy track. Although true, this is not as serious as would appear at first. Transport track is necessary in any case, and this can be laid on top of the 20-ft. ties for the excavator rails. Not more than 1 000 ft. of excavator track are needed per unit, inasmuch as during the progress of the work the rear track units are carried forward, so that the track creeps along in a longitudinal direction. A greater difficulty is presented by the lateral shifting of the track in cases where the excavator chain is not long enough to complete the excavation from one position. Normally, a gang of forty men would be kept busy attending to this work, but with the mechanical track-shifter used no manual labor is required. Fig. 5 shows a track-shifter at work.

The machine consists of a steel truss, 50 ft. long, the ends of which rest on swiveling trucks which ride on the track to be slewed. A jaw at the center of the truss picks up the track at the rails and moves it in a lateral direction, so as to form a kink in the line. Nothing is disconnected, the entire track is moved, including 20-ft. ties and the transmission masts attached thereto. In the next position one kink is straightened out, and a new one produced farther ahead, and so on until the desired position is attained.

Corresponding to the large excavation units used throughout the work and the haulage distance of 1½ miles on the average, the rolling stock also had to be of large capacities. About 900 side-dumping cars of 7 and 8½ cu. yd. capacity on 3-ft. gauge were used. The side-dump type especially allotted to the earth transport is shown on Fig. 6. These cars are tipped by one man operating a lever on the far side.

Earth-Filling Operations.—Fig. 6 shows the traveling machines which were used for placing the fill. These bank-building machines unite the functions of a multiple-bucket excavator and a belt conveyer. They weigh 240 tons, travel at a speed of 27 ft. per min., and climb grades as steep as 3⅓ per cent. They run on four rails laid on ties 20 ft. long, which also accommodate the rails for the earth transport trains. The latter dump their material into a trench about 8 ft. deep which the bank-building machine itself has formed. Two chains with ⅓-cu. yd. buckets pick the material up from this feeder trench, and drop it into a hopper which discharges on to a 36-in. belt running upward at a gradient of 20° with a speed of 300 ft. per min. From this belt, the material falls upon a second belt which travels along a boom and is reversible. These two qualities combine to permit covering a width of 125 ft. from one position. The maximum dumping height is 50 ft. The theoretical capacity is 1 000 cu. yd. per hour, corresponding to twice 25 bucket discharges per min.; that is, there were two bucket ladders, each capable of discharging 25 buckets per min. The actual quantities placed in an average 18-hour day were from 6 000 to 7 500 cu. yd. This output is governed more by the efficiency of the transport system than by the machine's performance, and could be doubled under favorable circumstances.

It has been mentioned before that, to shorten interruptions due to repairs and reduce the required stock of spare parts, units of the different machines were made interchangeable. This idea was applied not only to mechanical parts such as excavator buckets, chain links, etc., but also to the electric equipment. Thus, the bank-building machines were equipped with the same types of motors, as far as feasible, as the multiple-bucket excavators. The two bucket chains and the upper conveyor belt were driven by 50-kw. units each and the lower belt by a 22-kw. motor. Besides these, there were the drives for the two winches for raising and lowering the bucket chains, a drive moving the lower belt along the boom, the advancing drive, and drives for the two pumps which were in readiness to supply water for sprinkling the fill if required.

Comparison with Other Methods.—Having described the machinery introduced, it may be well to compare its performance with other methods of constructing embankments. The time-honored method of building up the fill by dumping earth from tracks which are jacked up by a gang of men as the work progresses, was out of the question. The red marl is an excellent material as regards water-tightness, inasmuch as no more seepage than 0.1 gal. per sq. ft. per hour has been observed, but it is rather easily saturated with water, and it then tends to flow and slump together. Under these circumstances the track work, which plays an important part in this method, in any case, would have made the undertaking very expensive; the frequent rains would have stopped the work for days; and progress would have been very slow.

Two reasons mainly argue against the hydraulic-fill method: First, the quantity of water required would have to be carried from the river in a pipe line several miles long; and, second, a reason which in itself was sufficient to make the contractors abandon the idea, the material was wet enough as it was, in fact, part of it had to be excluded on account of its saturation and corresponding lack of stability which would have imperiled the embankments. Moreover, this method requires auxiliary equipment, such as drag-lines for building the shoulders, derricks, flumes, booster pumps, etc., and there would have been extensive auxiliary operations.

Dumping the material from trestles would have been another method; but apart from the great quantities of imported timber required for 15 miles of embankment, and the necessity of raising, not only the earth but also the dead load of the locomotives and cars to high elevations, there is the prejudice against leaving trestle bents in the completed fill.

General Advantages of New Method.—It is believed that the method adopted combines the advantages of the other systems without suffering from their drawbacks. Of all applicable methods, it is the one which does not require equipment of any description to travel or operate on new fill; the bank-building machine was the only unit used, and it traveled on the firm undisturbed ground at the land side of the embankment.

The two principal requirements for a structure used for impounding water are stability and impermeability. Apart from intelligent design of the cross-section, these requirements can be fulfilled by providing proper distribution of the material according to its water-tightness and weight, proper compacting of the fill, and avoidance of planes of weakness.

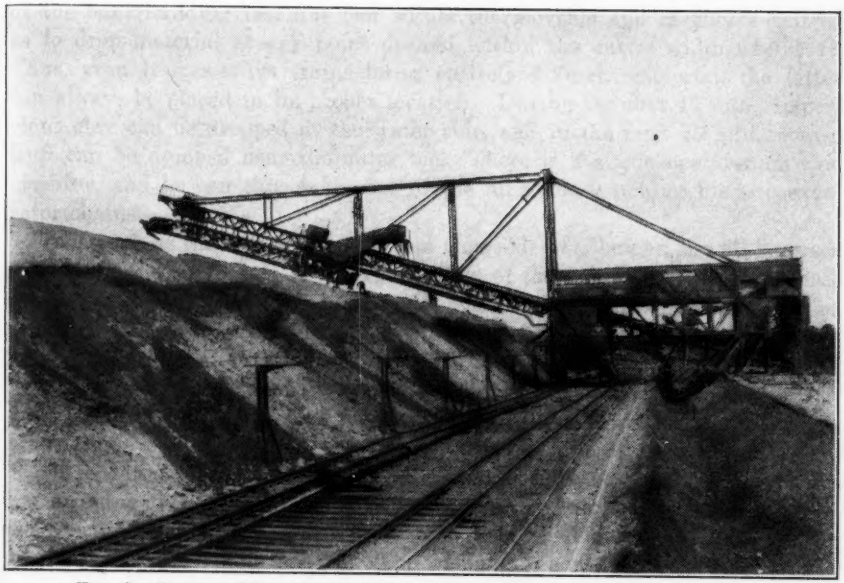


FIG. 6.—VIEW OF BANK-BUILDING MACHINE, SHANNON POWER DEVELOPMENT.

FIG. 7.—PLACING CONCRETE CANAL LINING ON STONE BED, SHANNON POWER DEVELOPMENT.

By pressing a contact button on the switchboard at his side, the operator of the bank-building machine can adjust the movable and reversible belt so as to drop material at any point desired within the entire width of 125 ft. Thus, even if successive trains bring entirely different material, the latter can always be placed in its proper location. During the first 15 min., impervious clay can be dropped at the water side, and in the next 15 min., coarse stuff can be dumped near the outer toe. There is a single operator on this machine, and he can thus take care of the fill while watching his two excavator chains.

Inasmuch as the material drops from a considerable height, the fill becomes very dense and compact. The tamping effect of the dropping stones is at least equivalent to that of a roller, and under some circumstances its effect will be greater. Moreover, there is the absolute certainty that every inch of fill, vertical and horizontal, has been compacted.

Rolled fills have a tendency to show what may be called "pancake" structure. Between well-compacted layers there occur looser strata which offer less obstruction to seepage. In the method described herein horizontal planes do not occur. When the lower conveyor belt is kept in a fixed position and the machine travels parallel to the canal axis, a longitudinal pile of earth is formed. Piles of, say, 5 ft. are built up side by side, and then the valleys are filled and new ridges formed. The whole dam consists of diamond-shaped prisms, so that, even if their contact planes did operate as weakness planes, the percolating water would have to climb one ridge after the other. Actually, the new material has been observed to pierce the old, and thus it forms a good bond. Moreover, the bank-building machines are equipped with sprinkler pipes, so that any desired consistency may be obtained.

The Finishing Work

Since care was taken to excavate the canal and fill the embankments to the proper level and slope, there was little dressing to be done. For additional safety the water faces received a special layer of clay puddle 1 to 2 ft. thick, which again was protected by rip-rap 12 to 16 in. thick. In the zone where water-level variations and wave action were to be expected, a concrete slab, $4\frac{3}{4}$ in. thick, was added as a further precaution. The unprotected banks and crown were re-soiled with the humus laid aside for this purpose in the beginning. For part of this work the bank-building machine could again be utilized; but part of it was done with the aid of swivel cranes. The latter also served for placing the rip-rap and concrete. Fig. 7 illustrates these operations.

Conclusion

A new method for handling large embankment building contracts has been explained. It is especially adapted to construction work involving great longitudinal extension. While it is mainly intended for handling lighter earths, it has also proved economical under severe conditions, in wet and viscous soil, and in a climate unfavorable to outdoor construction work. This fact is illustrated by the low price at which the work could be done; 1 cu. yd. of excavation and fill, including haulage, costs little more than 20 cents.

The work described in this paper was done by the firm of Siemens-Bau Union, as sub-contractors for the Siemens-Schuckert Works, Berlin, Germany.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

ICE AS AFFECTING POWER PLANTS*

FINAL REPORT OF COMMITTEE OF POWER DIVISION

At the Annual Convention of the Society held in Chicago, Ill., in July, 1923, a paper was presented to the Power Division by William T. Walker, M. Am. Soc. C. E., on the subject "Ice Problems of Hydro-Electric Plants." In this paper Mr. Walker pointed out the need of the study of ice problems and urged the formation of a Committee. At the conclusion of the discussion it was voted to appoint such a committee of the Power Division to study the subject of "Ice as Affecting Power Plants." The Committee appointed by Charles F. Loweth, Past-President, Am. Soc. C. E., (then President, Am. Soc. C. E.), consisted of the following members: Francis C. Shenehon, Chicago, Ill., *Chairman,* John R. Freeman, Providence, R. I., John L. Harper, Niagara Falls, N. Y., Daniel W. Mead, Madison, Wis., Louis C. Sabin, Cleveland, Ohio, and William T. Walker, Minneapolis, Minn.

The death of Mr. Harper in November, 1924, reduced the Committee to five. The geographical spread of the membership made meetings of the Committee difficult so that interchange of views has been by letter. The conception of the Committee of its best usefulness involved three elements: Observation, program-making, and compilation of facts as to existing practice. Consideration was given to the desirability of doing some research work, but none was undertaken.

At the Fall Meeting of the Society at Detroit, Mich., in October, 1924, a session of the Power Division was devoted to "Ice as Affecting Power Plants." Mr. Shenehon presented a brief report on the progress of work by the Ice Committee. An interesting talk was given by John Murphy, Electrical Engineer, Canadian Department of Railways and Canals, of Ottawa, Ont., Canada. Mr. Murphy's talk was illustrated by motion pictures and slides showing frazil formation and devices used to combat ice in hydraulic plants. A comprehensive discussion entitled "Ice Problems at Hydro-Electric Plants" was presented by Thomas N. Hogg, M. Am. Soc. C. E., Hydraulic Engineer of the Ontario Hydro-Electric Power Commission.† No activity of the Committee appears in the records of 1925.

* Presented at the meeting of the Power Division, New York, N. Y., January 17, 1929.
† *Canadian Engineer,* November 11, 1924.

In 1926 a search was authorized in the Engineering Societies Library under the following specifications:

(*A*) Ice jams in a river approaching a power plant, or jams in a river below, causing back-water rise and loss of head.
(*B*) Floating ice drawn into intakes or approach canals.
(*C*) Sheet ice forming in approach rivers, intakes, canals or forebays, including ice thrust on racks, gates, dams, and control works.
(*D*) Frazil as affecting intakes, control works, racks, and turbines.
(*E*) Snow, slush, or shell ice in ditches, flumes (as in the Sierras), intakes, control works, racks, and turbines.
(*F*) Combinations of Specifications (*A*), (*B*), (*C*), (*D*), and (*E*), in affecting power plants, forming on gates, racks, or dams.

Engineering treatments that have been devised to meet these troubles may be subdivided as follows:

Interception of sheet ice forming and breaking up in a head reservoir or lake (Specifications (*A*), (*B*), and (*C*)). This may be accomplished by the creation of slow current with consequent sheet-ice formation which serves as a barrier; or by intercepting works—as at Messina, N. Y.—formed of cribs and booms. It may also be accomplished by design of entrance of intake, canal, or forebay to by-pass the floating ice. The extent to which heat treatment may apply depends upon many conditions.

The physics of frazil (Specification (*D*)), including water temperatures, etc. (see works of Professor Howard T. Barnes, of McGill University, Montreal, Que., Canada); methods of combatting frazil; heat treatment; and forecasts of approaching trouble through surface temperatures.

Specifications (*E*) and (*F*) involve answers to the following questions: How far has the introduction of higher temperature water into forebays or collecting ponds lessened troubles in flumes or ditches? Will the housing of racks, with a temperature of 70° in the rack room, prevent the clogging of racks with frazil?

The search was completed in February, 1927. (See Appendix.)

In December, 1926, assignments were made to each member of the Committee on Ice as follows:

1.—Mr. John R. Freeman: The use of artificially produced heat, such as steam or warm air, to solve certain ice troubles in power plants.
2.—Mr. Daniel W. Mead: The solution of the problem of racks and their design to avoid ice troubles or to meet them.
3.—Mr. Louis C. Sabin: The ice conditions and troubles of the St. Marys River power plants and devices used in solving these troubles.
4.—Mr. William T. Walker: The ice troubles in the condensing water inlets of big steam plants and their solution.
5.—Mr. Francis C. Shenehon: The use of artificially produced heat, such as steam or hot air, to solve certain ice troubles in power plants.

In August and September, 1927, Mr. Shenehon visited hydro-electric installations in Norway and Sweden. In Sweden he visited the Porjus Development in Lapland north of the Arctic Circle and observed expedients used by the hydraulic engineers of these countries to meet winter conditions.

At the meeting of the Power Division in New York, N. Y., on January 17, 1929, Professor Barnes presented a paper on "The Constitution of Water and Nature of Ice,"* and Mr. Walker presented a paper on "Ice in the Intakes of Steam Power Plants."† The Committee on Ice as Affecting Power Plants, following this meeting, submitted its final report and requested dismissal.

Perhaps in the logical sequence of a treatise on ice the fundamental fact is the nature of water and its transformations under heat variation. Professor Barnes has treated this subject in his paper previously mentioned. He has pointed out‡ that,

"* * * water is an associated fluid, that its molecules are not simple H_2O molecules but molecular aggregates of complex form. Steam or water vapor consists of simple molecules which are called Hydrol. Water appears to be composed of double molecules called Dihydrol, while ice of triple molecules is called Trihydrol. From studies of the physical properties of water such as specific heat, density, and viscosity, it has been concluded that the associated water molecules of dihydrol contain varying proportion of the triple molecules trihydrol, and that the amount of the latter increases as the temperature falls from the boiling point. At the freezing point the proportion of trihydrol is estimated to be as high as 37% and at 100° C. about 16%.

"Roentgen as early as 1892 stated that water contained ice in solution above the freezing point, and more recent studies have shown the identity of these ice molecules with the trihydrol. Before congealing, the water contains ice as a solution carries a salt, and the freezing point is the temperature where the solution becomes saturated with the ice and ready to produce the solid when the necessary energy is abstracted to enable the solid to precipitate out. To get this energy to flow out of the solution a small temperature depression is necessary, which we have already described. That the supercooling is not more pronounced is due to the rapid rate at which ice is formed tending to keep up the temperature so long as any water remains to form ice. Water supercooled by laboratory experiments to many degrees below 0° C. is a highly supersaturated solution of ice, and readily precipitates on being agitated or by introducing a small particle of ice. It is exceedingly interesting to observe the nature of this first precipitation. The particles of ice as soon as they can be seen under a microscope are devoid of crystal form and appear as a true colloid in small disc-like particles. These flocculate and grow, passing through a crystal colloidal form to a true ice crystal. On allowing to grow slowly in quiet water they assume the form of a branching feathery snow crystal of most beautiful form. In rapidly moving water the crystals never become large, but are broken up and assume more and more a mass of broken spicules or spines. In Nature, we find this first precipitation of ice in water appearing as a cloudy formation throughout the mass of the water. This is the beginning of the fine ice noticed in rapids or open water and called frazil or needle ice. It is the form which gives the most trouble in power house operation, since it is produced all through the mass of the water and is of little or no buoyancy. It is readily carried into the racks and wheels, and adheres to the surface of machinery already cooled below the freezing point by the contact of the chilled water flowing through."

The illuminating statement that the flowing fluid water of a stream as it approaches the freezing point may contain more than one-third of its volume in the trihydrol or incipient ice form prepares the mind for a clearer under-

* *Proceedings*, Am. Soc. C. E., March, 1929, Papers and Discussions, p. 685.
† *Loc. cit.*, p. 689.
‡ "Ice Engineering," Renouf Pub. Co., Montreal, First Edition, April, 1928, pp. 6-7.

standing of the amazing phenomenon of the appearance of swarms of frazil, springing into existence when the agitated water drops less than 0.01° cent. below the freezing point. "Before congealing, water carries ice as solution carries salt!" Liquid ice is, therefore, a proper physical conception. The emergence of visible cloudy colloidal ice in water is likened by Professor Barnes to the visibility of mist emerging in an aqueous vapor. The delicate balance in the temperature of water passing from the liquid form to the colloidal and particle form lies safely within 0.01° cent. on either side of the zero line. If water is held to a temperature of 0.01° cent. no ice will form. Ice will form rapidly with a water temperature of — 0.01° cent. and agitation.

The physical constants of ice are essential in this discussion. The density of clear ice may be taken as 0.92 and its weight at 0° cent. as 57.4 lb. per cu. ft. Where air is entrained in ice it is of course lighter. Water is known to increase in density as it is cooled until its maximum is reached at 5° cent. It then expands until its density at the freezing point is the same as its density at 8° cent.

The latent heat of the fusion for ice is 143.5 B. t. u. This means that the transformation of a solid pound of ice takes 143.5 times the heat necessary to raise the temperature of liquid water itself 1° Fahr.; and with 60% efficiency and coal having a heat value of 14 000 B. t. u., 1 lb. of coal will melt 59 lb. of ice; and 1 lb. of petroleum will melt 98 lb. of ice. The heat necessary to raise 1 lb. of water 1° Fahr. is the British thermal unit. This means that with coal of 14 000 B. t. u., 1 lb. of coal will raise the temperature of 14 000 lb. of water 1° Fahr. As a temperature rise of 0.1° will avoid freezing, 1 lb. of coal, therefore, used with 60% efficiency, will treat 1 340 cu. ft. of water. If this water is flowing through a steel siphon at the rate of 40 cu. ft. per sec., 1 ton of coal will serve for tempering to this extent this volume of water for 18 hours. Stated in another way if the temperature of the ditch water approaching a siphon is 32°.0, the addition of 0.1 cu. ft. per sec. of water at 70°, will raise the total volume to 32°.1 Fahr. and will be a safeguard against freezing.

The mechanical equivalent of heat may be taken as 1 h.p.-hour = 2 546 B. t. u. If this be taken in the form of electric current transmitted to the point of use at 70% efficiency this becomes 1 h.p.-hour = 1 780 B. t. u. Therefore, 1 h.p. will serve to raise the temperature of 28.5 cu. ft. of water 1° in 1 hour. The use of mechanically produced heat or electric current must be restricted to the maintenance of temperature of perhaps 32°.1 Fahr. in the metal of racks, gates, or turbine wheels, in order to avoid the adhesion of frazil.

The latent heat of the vaporization of ice is the sum of the latent heats of the melting of ice and the vaporization of water. This is about 143.5 + 1 076.5 = 1 220 B. t. u. The bearing of this constant has to do with evaporation from ice.

The specific heat of water is unity at 17°.5 cent., or 63°.5 Fahr. At 0° cent. the specific heat is 1.0094 and at 10° cent. it is 1.0023. The British thermal unit, therefore, applies strictly to water when its specific heat is

unity. About 1% more heat is needed to raise the temperature of water at 0° cent. than at 17°.5 cent.*

The thermal conductivity of snow and ice is relevant in a study of water conditions in power streams. The thermal conductivity of ice, snow, and water is expressed in calorie-gramme-second units. For comparison the thermal conductivities of familiar metals are as follows:

Materials.	Thermal conductivity, in calorie-gramme-seconds.	Materials.	Thermal conductivity, in calorie-gramme-seconds.
Ice	0.005	Iron	0.11
Snow	0.0003	Silver	1.00
Water	0.0013	Steel	0.06
Aluminum	0.48	Zinc	0.26
Copper	0.89		

The blanket value of a snow-and-ice cover in a fore-bay pond is obvious.

The coefficient of expansion of ice is given by Professor Barnes as about 0.000 089. In the American Civil Engineers Handbook, Fourth Edition, a value ascribed to Ganot of 0.000 052 is given.

Professor Barnes has discussed† other physical elements, such as plasticity, elasticity, viscosity, strength, color, refractive index, electrical properties, and bacteriology. It appears needless and undesirable to include in this report a discussion of the physics of ice. The Committee on Ice in this report has utilized this work of Dr. Barnes on "Ice Engineering" to the fullest extent.

Surface Formations

Sheet Ice.—The formation covering the water surface. In its simplest form, it is transparent even when a foot or more thick.

Cake Ice.—Solid sheet ice broken up by wind action, or other forces.

Border Ice.—Sheet ice forming in marginal bays where the water is shallow and the current slow.

Shore Ice.—The formation, along the banks of a stream with brush, drift-wood, or débris incorporated.

Windrow Ice.—A consolidated aggregation of cake ice with edges up.

Honeycomb Ice.—Sheet ice rotting in place and eventually breaking down into long crystal needles.

Slush Ice.—Unconsolidated snow floating on the water surface after a snow-storm; or water-penetrated snow, covering and depressing sheet ice by its weight.

Snow Ice.—Consolidated slush ice.

Shell Ice.—Thin sheet ice forming and breaking up in such streams as the Ohio River above Louisville, Ky.

Floe Ice.—Bergs or masses from spray or water action.

Middle Formations

Frazil or Needle Ice.—The swarming mass of spicules forming throughout the full depth of open, flowing, agitated, supercooled water. It is the precipi-

* "Ice Engineering," by Howard T. Barnes, 1928, pp. 38-40.

† *Loc. cit.*, pp. 40 *et seq.*

tation in needle form of the solid ice, out of the liquid trihydrol element in the water.

Hanging Ice.—An agglomeration of frazil, anchor ice, slush ice, shell ice, and cake ice, forming on the bottom of sheet ice at the foot of a stretch of open swift-flowing water.

Bottom Formations

Anchor Ice.—A formation crystallizing on submerged objects, such as boulders, perhaps growing by the agglomeration of frazil.

In considering the adverse effects of ice on power-plant operation, the discussion may be assembled under the following headings.

A.—Lessened stream flow from a lake or reservoir because of ice formations in the outflow channel.
B.—Ice accumulations in reservoirs.
C.—Ice accumulations in approach canals and forebays.
D.—Lessened head from back-water caused by ice formation in outflow channels down stream from a power plant.
E.—Difficulties of gate operation in sub-zero weather.
F.—Ice formations or jams on spillway dams tending to block them.
G.—Ice formations on reservoir structures with varying levels in the reservoir.
H.—Ice thrust on hydraulic structures.
I.—Ice clogging of racks.
J.—Ice clogging of valves and turbines.
K.—Ice on the intake screens of steam plants.

It should be clearly understood first that the successful operation of a water power plant is not a matter of latitude or of the intensity of cold. The great State-owned power plant at Porjus, Sweden, lies 60 miles north of the Arctic Circle. In the deep lakes or fjords forming the reservoir and forebay pond of this plant, ice forms 3 ft. thick.

The Swedish engineers have built, still farther to the north, a multiple-arch dam—one of the most delicate of hydraulic structures—to conserve and regulate the flow of the Lule River. This river has an ultimate regulated minimum flow of 7 000 sec-ft. The Porjus development utilizes a fall of 184 ft., and has a present installation of 56 500 kw. and an ultimate capacity of 113 000 kw. The current is used for the electrification of the railroads of that region, for the operation of the iron mines at Kiruna and Gallivare, and for chemical industries. The forebay reservoir is deep, with a thick ice-and-snow blanket that keeps the water warm. The water is conducted through a deep, wide, slow-flowing open-rock cut intake canal, then through 1 700 ft. of tunnel in rock to a roof-covered distribution reservoir; then in vertical shafts in the rock, to the turbines in a power house carved out of solid granitic rock. The discharge tunnel 4 200 ft. long in rock leads to the lower levels of the river. The water being carried through rock from intake to tail-water freezing is completely avoided, even in extremely cold weather.

The same situation exists in the great Rjukan development on the Mana River in the mountains of Norway. Here, the water is conducted from a deep reservoir through rock-hewn tunnels to the three power plants; but in a development at Svaelgfoss farther down on the same river, ice troubles are

severe. It was said that 10% of the current generated in sub-zero weather was used to solve ice troubles. The fundamental fact in the expectation of ice trouble is conservation of the heat in the water as it exists in deep reservoirs. In such deep reservoirs the bottom water, even in mid-winter, is not far from the temperature of the maximum density of water, 39.2° Fahr.; and from that temperature it varies to about 32° at the bottom of the ice cover. The snow-covered ice acts as an insulator against the escape of heat and the earth crust temperature radiates from the bottom. Doubtless some heat from the sun penetrates the snow-and-ice insulation, and the freezing of the water on the under side of the sheet ice releases some heat by the process of molecular change. In deep, slow-flowing water, the rotation brings up the bottom strata and the mean temperature of the mass may be as low as 35° Fahr.

The obstruction of the outflow from a reservoir by ice formation in the stream is illustrated by the St. Clair River at the foot of Lake Huron. The normal summer fall in the 40 miles from Port Huron, Mich., to Lake St. Clair is 5.5 ft. When the sheet-ice cover forms in January, this 5.5 ft. of potential energy must serve the friction of the ice perimeter, in addition to the bottom perimeter of the open season. The result is a slower current flowing with lessened cross-sectional area. The outflow volume is at times only one-half the normal summer outflow. This was analyzed by Mr. Sabin, in the report of the Chief of Engineers, U. S. Army, for 1902. In this report, also, the lessened outflow of the great power stream, the St. Lawrence River, is analyzed by Mr. Shenehon, and in the report for 1900 he analyzes the lessened outflow of the Niagara River. The full water-power development of the St. Lawrence River, converting it into a series of slack-water pools where now turbulent rapids exist, may enlarge the winter flow.

The ice formations in a great reservoir like Lake Erie, which pass into the Niagara River in a procession of cake ice and floe ice, cause considerable trouble in the power plants at Niagara Falls. This ice may run up to February 1, when the foot of Lake Erie freezes over and forms a barrier against the entrance to the river of the drift ice of the open middle part of the lake. After the break-up of the sheet-ice barrier—perhaps in late March—the ice may run until mid-May. This run is largely cake ice with some ice floes, which ground in 8 ft. of water. In a paper entitled, "The St. Lawrence Waterway to the Sea",* by Mr. Shenehon, a project of breakwaters is suggested to minimize troubles in the Niagara River from Lake Erie ice. At the head of the Gota River in Sweden "an ice boom at the outlet of Lake Vanern prevents snow and ice from entering the river during northerly wind, reducing ice trouble."†

The entrance of floating ice into approach canals and forebays leads to many complications and difficulties. Where an approach canal is long, with a definite open-season loss of head, this loss is accentuated by the formation of sheet ice; or by packing with cake ice moving less rapidly than the water; or by the jamming of the ice. The by-passing of river or reservoir ice at the

* *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 497.

† "Utilization of Water Power and Inland Navigation," by A. Ekwall, World Power Conference, 1926, Section 27, p. 7.

entrance to the canal or forebay is the fundamental solution. The principles of by-passing ice have been discussed* by Benjamin F. Groat, M. Am. Soc. C. E. The practice he suggested and utilized for many years is the separation of the surface water for the transportation and by-passing of ice in the direction of the flow of the stream, while the power water is drawn from the middle or bottom strata, preferably at right angles to the stream direction. Guides or vanes in the bottom or gathering tubes may facilitate the cross flow of the lower strata. The power water may pass under a curtain-wall built parallel to the direction of the stream. The velocity of the directed power water must be low enough to avoid pulling the ice under by suction.

The approach to the long penstocks of the Ontario Company at Niagara Falls, built early in the Twentieth Century, illustrates the principle. The surface water carrying the buoyant ice approaching from the head of the rapids is carried past the curtain-wall and discharges with a depth of 2 ft. to 3 ft. over weirs on the gathering-basin walls. A curtain-wall parallel to the Niagara River current guards the entrance to the long Chippewa-Queenston Canal, and the principle of deep-water draft is used, with gathering tubes contemplated. The approach to the ¾-mile canal of the Niagara Falls Power Company, is guarded by booms and by ice formations worked out by the late John L. Harper, M. Am. Soc. C. E., to by-pass the surface ice over the American Fall. At Massena, N. Y., on the St. Lawrence, ice diversion follows the principle of sub-surface intake. At Marseilles, Ill., on the Illinois River, the curtain-wall is utilized less effectively. When ice is running it is necessary to keep a gang of men at work to prevent the cake ice from passing under the wall. At best, some buoyant ice will enter canals and some ice will form in canals, and sluicing is necessary at the power house. A power sluice was used by the Niagara Falls Company. The present device, however, is the endless chain elevator method of mechanically lifting the ice from the water and dropping it into a chute.

It is best, however, to avoid floating ice and frazil by drawing power water from a deep ice-and-snow-covered pool. Mr. Freeman has proposed a dam at the head of the rapids at Niagara Falls below the intakes of the upper three power plants to raise the water 10 ft. This would mean slower current and possible ice cover in the river between Lake Erie and the Falls. When a current exists approaching a power canal, the use of booms may aid in the formation of a surface sheet. This is the practice in the Gota River, Sweden, at the 115 000-kw. power plant, in Trollhättan.

The lessened head caused by a back-water rise is illustrated by the conditions in the Grass River down stream from the power plant at Massena. At the foot of the Lachine Rapids of the St. Lawrence River at Montreal, Que., Canada, a winter rise of 12 ft. is due to ice cover in the river down stream and the formation of huge hanging dams of frazil, anchor, and débris ice. The same cause sometimes obliterates the normal 11-ft. summer fall of Rapide Plat at Morrisburg, Ont., Canada, and Waddington, N. Y. The back-water here has been accentuated at times by the farmers along the river swinging

* "Ice Diversion, Hydraulic Models, and Hydraulic Similarity," *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1138.

a field of sheet ice, formed in a bay, across the current to form a bridge. In the Illinois River below the power plant at Marseilles, the formation of windrow ice and hanging dams at times obliterates the 11-ft. fall. The complete power development of a river by replacing rapids with slack-water deep pools will remedy the back-water. A deepening and straightening of the channel of the Grass River below Massena bettered the back-water conditions.

The difficulties of gate operation under ice conditions are met by the use of live steam—which may be generated in a boiler electrically fired. The use of warm compressed air bubbling against the up-stream side of the gates helps in part and avoids ice thrust. One of the 65-ft. roller gates of the Solbergfoss Plant of the City of Oslo, Norway, is electrically heated for emergency winter operation. This type of gate is readily warmed by a contained heating unit.

The spillway dam appurtenant to a power plant in a stream of heavy ice runs, especially if it be also a flood stream, needs the longest uninterrupted crest length. The roller gate, 20 m. long, previously mentioned, has many advantages over the relatively short-span Taintor or Stoney gates. Better yet are wicket gates which leave when down a clear crest line. The avoidance of ice formation on the crest of the hollow concrete Norrfors Dam is accomplished by steam pipes passing through the chambers, from a boiler electrically fired. At the Hammerfossen Dam the piers instead of having cut-water prows, have a flat up-stream end, the water cone formed serving effectively to avoid the lodging of logs or ice. These two dams are in Sweden. Robert Fletcher, M. Am. Soc. C. E., has painted* a vivid picture of the combination of ice and flood:

"At Summer's Falls a rocky barrier extends so obliquely across the river that its length is nearly double the direct width of the river. About 50 years ago at this site there was a dam, a canal lock and approach canal for river boats, and a very large saw-mill running seven saws. A spring flood brought down ice which jammed and froze; a second flood increased the jam, piled the ice high on the dam and against the banks, and finally carried the mill down stream, and wrecked the dam and lock, which were never rebuilt. The writer has seen an impressive picture of blocks of ice up to 4 ft. thick (a man standing beside one) wedged together over an extent of many acres, on one side of the Lower Yellowstone Dam, in Montana, as the result of a high flood. This suggests in part the possibilities of destruction by a spring flood carrying ice."

Ice formation in mountain reservoirs in February with continuous pull-down involves the spalling and disintegration under freezing of concrete dams. In a paper† by Fred O. Dolson and Walter L. Huber, Members, Am. Soc. C. E., remedies for frost action are stated. For a multiple-arch dam it would appear simple and not too expensive to carry down an insulating wall or diaphragm on the down-stream side, so that the bays are enclosed. Maintaining, by salamanders or otherwise, a temperature of 33° Fahr. within the chamber should safeguard the up-stream face against ice adhesion. Heat treatment in such a case will be more effective than super-waterproofing. The

* *Transactions*, Am. Soc. C. E., Vol. LXXII (1918), pp. 1161–1162.

† "Multiple-Arch Dam at Gem Lake on Rush Creek, California," *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 713.

heating of the hollow Ambursen type dam at Norrfors, Sweden, has already been mentioned.

Ice thrust on the up-stream faces of dams, unless intercepted, involves great forces. Professor Barnes* has given the following values:

Ice thickness, in inches.	Thrust, in pounds per linear foot.
6	28 800
8	38 400
10	48 000
12	57 600

These values are based on a crushing strength of 400 lb. per sq. in. Quoting from Mr. George G. Bell, the ice thrust is given for 18 in. as 43 200 lb. per lin. ft. Mr. Bell concludes that a dam exposed to full ice pressure should be designed to resist from 43 000 to 48 000 lb. per lin. ft.†

A paragraph on "Ice Thrust" in the American Civil Engineers Handbook‡ is as follows:

"Ice pressure acting at the top of the dam is sometimes required to be taken into account. John D. Van Buren in 1895 advocated the use of 40 000 lb. per lin. ft. The board of experts for the Quaker Bridge dam (report of Aqueduct Com., 1889) recommended that the ice pressure should be taken at 43 000 lb. per lin. ft. of dam. For the design of the Kensico Dam 47 000 lb., for the Cross River dam 24 000 lb., and for the Croton Falls dam 30 000 lb. per lin. ft. of dam have been used. These widely differing specified values of the ice pressure are mostly based on the crushing strength of the ice, but it is obvious that before pressures of such magnitude could be developed, the ice sheet would fail as a long column; moreover the creeping of the ice up the generally sloping shores of the reservoir will prevent any considerable thrust upon the dam. It can be shown (Trans. Am. Soc. Civ. Eng., Vol. 75, p. 164) that the consideration of ice pressure of such intensities as quoted above would result in absurdly large dimensions for low dams while the dimensions of high dams would be only slightly affected. It is believed unnecessary to consider ice pressure in the design of dams unless the reservoir is very short in the direction perpendicular to the axis of the dam and the sides slope abruptly."

The ice thrust assumed in the design of the Lule River (multiple-arch) Dam in Sweden above the Arctic Circle where the temperature falls to 50° below zero Fahrenheit and ice becomes more than 3 ft. thick, was less than 40 000 lb. per lin. ft. It is desirable to maintain by hand-sawing a gap in the ice sheet parallel to the dam, or to utilize bubbling compressed air. Even where wicket dams are used to keep up the winter head an open slot in the ice sheet avoids thrust.

Where metal racks reaching above the water surface are exposed to sub-zero weather in frazil-forming streams, the probability of complete blocking is great. Wherever this probability exists the racks themselves, or their supporting members, or both, should be designed to support the full load of the static head. Where the racks are enclosed in a heated rack room or rack chamber, it is still desirable to design the supports to carry the full static head, since a collapse means a long shutdown for repairs. The heating of the racks in some form so that ice will not fasten to them is desirable. The simplest form

* "Ice Engineering," p. 214.

† *Proceedings*, Maine Soc. of Civ. Engrs., Vol. I, 1911, p. 41.

‡ Fourth Edition, p. 709.

is the heated rack chamber made by decking over the rack space. Heat may be supplied best (as it is done in the Norrfors Plant in Sweden) by providing a conduit that carries the hot air from within the generator casing to the rack chamber. A curtain-wall is desirable to by-pass floating ice. In some rack rooms where the air temperature is but little above 32° Fahr. no trouble has resulted. However, in rivers, such as the Gota River in Sweden, where frazil forms and agglomerates and comes to the racks as huge slushy masses it is desirable to utilize additional safeguards. The fine screens are removed during frazil periods. The remaining coarse racks have clear openings between bars of 6 in. or more, and, in addition, electrical heating is ready for emergency use.

The Chief Engineer of Water Power in Sweden, Mr. A. Ekwall, estimates the electric current consumed in heating the racks as that coming from a 2-m. fall of water. Where the operating head on a plant is 20 ft., the energy consumed in heating the racks is 30% of the output; while with a head of 60 ft., it is but 10 per cent. The greatest safety against ice troubles lies in enclosed heated rack chambers, wide spacing in racks, and provision for emergency heating of racks. It should be noted that the use of electrical current is occasional and is likely to be at off-peak times. Should the racks be placed so that they may be flooded with sunshine, the condition is better still. The avoidance of night radiation and the absorption of heat by daylight are the ideal conditions. The complication of anchor ice rising in masses from the bottom, carrying sand, gravel, and boulders, and moving below the surface, complicates by-passing, reliance on curtain-walls, and the removal of the lower level racks.

The clogging of valves and turbine casings by massed frazil, slush ice, or anchor ice, is perhaps present in low-head plants with small units on horizontal axes. Where such clogging occurs, the use of live steam is effective in restoring operation. Mr. Hogg suggests concrete turbine casings "with series immersion electric heaters."* The use of electric current in the form of the water rheostat has been effective to break up ice masses. Bubbling warm air is also used. Professor Barnes advocates the use of thermit.†

It is understood that compounds more effective than thermit are in course of development. Other forms of chemicals are used, such as calcium chloride. Flooding with heat rays is also in use. It is perhaps desirable to accentuate the higher effectiveness of preventive measures to avoid clogging, rather than the surgical methods of breaking up the ice after a shut-down. So a little heat is needed—perhaps, a few thousandths of a degree—to avoid ice adhesion to metal structures, that the best practice is mild heating of dangerous parts.

Ice troubles in the screens of condensing-water intakes in steam plants and the remedies discussed by Mr. Walker‡ who points out the superior economy of re-circulating the condensing water. The breaking of ice jams in power streams is an art in itself, involving the use of dynamite, thermit, salt, ash, sand, and gravel.§

* *The Canadian Engineer*, November 11, 1924, p. 497.

† In *Engineering Journal*, The Eng. Inst. of Canada, November, 1926, Professor Barnes discusses the nature of thermit ice reaction.

‡ *Proceedings*, Am. Soc. C. E., March, 1929, Papers and Discussions, p. 689.

§ *Engineering Journal*, The Eng. Inst. of Canada, November, 1926, contains descriptions by Professor Barnes of the "Breaking of the Allegheny Ice Gorge of March, 1926."

In concluding this report the Committee may summarize by saying that ice troubles are consolidated cold troubles; that heat is the instrument to prevent trouble; that heat is present in the water itself conserved in deep ice-and-snow covered reservoirs; that daylight and sunshine are great heat resources; that the hot air inside the generator casings of power plants is a by-product to be utilized perhaps more effectively than heretofore; that discharged condensing water in steam plants may in part be re-circulated to temper the intake water; and that electric energy itself, in its thermal capacity, is a facile instrumentality—and not too wasteful at off-peak hours of the night or at daybreak. It is hoped that the inadequacy of this report may be compensated by its provocative quality. If it arouses discussion it will serve a useful purpose. Each power plant is a problem in itself. Narratives of experiences, the failures and successes of operators, will add much to the perhaps too scanty survey of the subject.

FRANCIS C. SHENEHON, *Chairman*
JOHN R. FREEMAN,
DANIEL W. MEAD,
LOUIS C. SABIN,
WILLIAM T. WALKER,
Committee.

January, 1929.

APPENDIX

BIBLIOGRAPHY OF ICE IN HYDRO-ELECTRIC PLANTS

1881—Francis, J. B. Anchor Ice. 1881. (In *Transactions*, Am. Soc. C. E., Vol. X (1881), pp. 192-194; part of Presidential Address).
Discusses the nature of anchor ice and its interference in the operation of water power plants.

1882—Report on Ice Harbor at the Head of Delaware Bay. 1882. (In Annual Rept., Chf. of Engrs., U. S. A., 1882, Pt. 1, pp. 785-790.)
Several paragraphs on ice pressure, followed by tables on crushing strength of ice, from experiments at Frankford Arsenal, Philadelphia, Pa., and at Fort Tompkins, Staten Island, N. Y.

1884—Ludlow, William. Observations on the Crushing Strength of Ice. 1884. (In *Proceedings*, Engrs. Club of Philadelphia, v. 4, pp. 93-99.)
To determine the resistance necessary to be provided against ice in constructing piers for an ice harbor at the head of Delaware Bay.

1891—Dumble, J. H. Some Observations on the Expansion and Contraction of Ice on Canadian Waters. 1891. (In *Transactions*, Canadian Soc. C. E., v. 5, pp. 270-308.)
Experience of the effect of ice movement on bridge piers.

——Abstract. Movement of Bridge Piers by Expanding and Contracting Ice. 1893. (In *Engineering News*, v. 29, pp. 41-42.)

——Editorial. 1893. (In *Engineering News*, v. 29, p. 37.)

1893—Failure of Small Dam. 1893. (In *Engineering Record*, v. 27, p. 400.)
Failure of dam by ice belonging to Des Moines Water Power Co., during low-water period. Dimensions of dam not given.

1894—Freeman, John R. Hoisting Apparatus of the Canal Head-Gates at Sewall's Falls, N. H. 1894. (In *Transactions*, Am. Soc. C. E., Vol. XXXII (1894), pp. 278-294; Discussion, pp. 294-332.)
Pages 295, 304, 306-320 contain data from actual experience on the formation of anchor ice on canal head-gates.

1897—Hudson, H. B. Effects of Mississippi River Floods on New St. Anthony Falls Dam. 1897. (In *Engineering News*, v. 37, p. 290.)
Failure of St. Anthony Dam, due to ice. Gorge formed 20 miles up stream, broke. Dam approximately 250 ft. long, of which 110 ft. were swept away by ice pressure. Photographs of the dam, before and after failure.

1898—Leonard, R. W. Masonry Pier Moved by Ice and Replaced. 1898. (In *Transactions*, Canadian Soc. C. E., v. 12, pp. 131-134.)
The thrust of the ice was about 11 tons per sq. ft., considering the ice 3 ft. thick and striking the pier obliquely on a width of 5 ft.

1899—Failure of a Minneapolis Dam by Ice Pressure. 1899. (In *Engineering Record*, v. 39, pp. 542-543.)
Plan of Minneapolis Dam given in diagram; width of dam was 12 ft. at base and 5 ft. at top; 250 ft. long and 18 ft. high. Dam owned by St. Anthony Power Co.

——**Rickey, W.** Failure of Dam at Minneapolis Due to Previous Weakening Through Ice Pressure. 1899. (In *Engineering News*, v. 41, p. 307.)
Illustration showing trench cut in ice along face of the dam. Dimensions of dam, 535 ft. long, controlling falls of the Mississippi River, at Minneapolis. Sectional view of dam and mill of the Minneapolis Mill Co.

——**Severe Test of Bridge Masonry.** 1899. (In *Engineering Record*, v. 40, p. 554.)
Effect of a mass of ice sweeping against a bridge, carrying away a span and moving a pier off its foundations.

1903—Cairns, R. A. Wigwam Reservoir Masonry Dam of Water-Works of Waterbury, Conn. 1903. (In *Engineering News*, v. 49, pp. 418-419.)
General description of the dam, including dimensions, architectural design, with photograph and plan view. Conditions existing up stream and general description of surrounding territory, character of soil, water movements.

1905—Canadian Niagara Power To-day. 1905. (In *Electrical World*, v. 45, pp. 17-20.)
Illustrates a set of iron ice racks, partly above and partly below the water level at the intake.

1906—Barnes, H. T. Ice Formations with Special Reference to Anchor Ice and Frazil. 1906. (Pub. by Wiley, N. Y., 260 pp.)
Discusses physical laws governing the transfer of heat, physical constants of ice, formation and structure of ice; the nature of sheet, frazil, and anchor ice; precise temperature measurements; theories to account for frazil and anchor ice and methods of overcoming the ice problem in engineering work.

——**Patton, W. M.** Practical Treatise on Foundations. 1906. Second Edition. (N. Y., Wiley, pp. 49, 107-113.)
Gives brief description of several cases showing effect of drift ice on the stability of piers.

1908—Dibos, M. Glaces de Mer, Glaces de Rivières, Manoeuvres de Preservation et de Deglacage. (Protection from breaking of sea and river ice.) 1908. (In *Memoires et Traveaux* de Société Ingénieurs Civils de France, 1908, Pt. 2, pp. 860-923; Abstract, *Minutes of Proceedings*, Inst. C. E., Vol. 179, pp. 441-442.)
Considerable space is devoted to descriptions of the various forms of ice and the phenomena of freezing. Illustrated descriptions are given of protective mechanism designed by an officer in the Russian Navy. The conclusions reached by the Congress of Navigation at St. Petersburg, regarding the best means of protecting structures from ice, are given. The oxy-hydrogen flame is suggested as a means of thawing grooves in solid river ice. Superficial and submerged charges of dynamite, melinite, black powder, or Favier safety powders may be used. Their advantages are discussed.

1911—Harrison, C. L. Provision for Uplift and Ice Pressure in Designing Masonry Dams. 1911. (In *Transactions*, Am. Soc. C. E., Vol. XXXVII (1897), p. 142.)
The question of uplift discussed with reference to types of construction for bridges and dams. Ice pressure on reservoirs and the necessity of considering this factor in connection with bridge or dam design.

1912—Failure of Dam from Ice Pressure. 1912. (In *Engineering News*, v. 65, p. 681.)
Failure of dam, 150 ft. long, 21 ft. high, and 15 ft. wide, belonging to Port Henry Light & Power Co., Rockport, N. Y. Ice, 3 ft. thick.

——**Partial Failure of a Masonry Dam, Caused by Ice Pressure.** 1912. (In *Engineering Record*, v. 65, p. 94.)
Partial failure of stone dam, on Saranac River, N. Y., due to ice pressure. Dam, 152 ft. long and 45 ft. high.

——**Wall, E. E.** Ice Trouble at the St. Louis Water-Works. 1912. (In *Engineering News*, v. 67, pp. 212-213.)
Discusses the use of a claimshell bucket to remove ice from the intake of the water-works plant.

1914—Barnes, H. T., Hayward, J. W., and McLeod, N. M. Expansive Force of Ice. 1914. (In *Proceedings*, Royal Soc. of Canada, Series 3, v. 8, Section 3, pp. 29–48.)
Table of coefficient of expansion of ice; observations made by the authors on the ice in the Montreal filtration chambers, 1914; conclusions as result of preliminary study of ice expansion, it is found that: (1) the crushing strength of ice is most probably 400 lb. per sq. in.; (2) an ice block will yield under pressure at approximately 200 lb. per sq. in., which is probably due to the slipping of the crystals; (3) an ice sheet will form cracks on the upper and under surface due to unequal strain; (4) that a permanent expansion may result if the cracks become filled and frozen; (5) according to the most trustworthy results of other observers, the ice frozen to concrete develops its full crushing strength, and the tensile strength of ice is under 200 lb. per sq. in.

——**Fuqua, P. D.** Breaking a Heavy Ice Jam in the Great Miami River. 1914. (In *Engineering News*, v. 71, pp. 997-998.)
Describes the diversion of a serious ice jam from bridge and dam at Hamilton, Ohio, by setting off large explosive charges. Brief data are given on the damage done to structures by ice scour.

——**Hendry, M. C.** Ice Troubles in Hydro-Electric Plants. 1914. (In *Canadian Engineer*, v. 27, pp. 693-694.)
Discusses the nature of sheet, frazil, and anchor ice and the effect of each type on the winter operation of hydro-electric plants.

——**Holtwood Dam Weathers Early Ice Run in Susquehanna River.** 1914. (In *Engineering Record*, v. 69, p. 421.)
The Holtwood Dam is said to have had a deterrent effect on the ice jams in the river below.

——**La Motte, A.** Dynamiting Ice Jams. 1914. (In *Engineering Record*, v. 69, p. 365.)
An effective means of breaking river ice is described. Submerged charges are considered most effective.

——**Winter Operation of Stoney Gates.** 1914. (In *Engineering Record*, v. 70, p. 691.)
Describes the application of steam conduits to the large dam gates at the Shawinigan Water & Power Co.'s hydro-electric plants on the Maurice River. Diagrams show the extent of ice formation on piers and gates.

——**Barnes, H. T.** Crushing Strength of Ice. 1914. (In *Transactions*, Royal Soc. of Canada, Series 3, v. 8, Section 3, pp. 19-22.)
C. A. Mees concluded that the crushing strength of ice might reach as high as 800 lb. in a short column. He says, however, "Wherever the effects of ice expansion have been carefully observed, it has been noted that bending takes place, and this may be expected because we have a long column effect. On this account assuming one-half the crushing strength may be developed before bending, the maximum thrust may equal about 400 lb. per sq. in." The author states: "Probably the best experiments have been made by Mr. George G. Bell; if we neglect his highest result, which appears to be too high, we find values more consistent, ranging from 358 to 783 lb." Details of author's experiments are given.

1915—Allner, F. A. Ice Fighting Is Systematized at Holtwood Hydro-Electric Plant. 1915. (In *Engineering Record*, v. 72, pp. 66-68.)
Describes the precautions taken at the Susquehanna River plant to prevent shut-down of main and exciter units due to frazil and anchor ice. Steam is used to warm turbine chamber.

——**Allner, F. A.** Severe Frazil Ice Attack at Holtwood Plant Does Not Disturb Service. 1915. (In *Engineering Record*, v. 72, pp. 113-115.)
Some detailed information on the first two onsets of frazil ice and the damage it incurred; at the time of the third occurrence the measures for protection had been put into practice. The situations are compared.

——**Bligh, W. G.** Dams and Weirs. 1915. (Am. Tech. Soc., Chic., 206 pp.)
Discussion concerning the necessity of considering ice pressure in designing dams, bridges, and reservoirs.

——**Smith, C. W.** Construction of Masonry Dams. (McGraw-Hill, N. Y., 272 pp.)
The subject of ice thrust is discussed on pp. 117-126, giving specific example of dams and reservoirs which have been successful and others which have failed.

——**Use of Special Gates to Prevent Ice Formation at Intake Screens.** 1915. (In *Electrical World*, v. 66, pp. 1030-1031.)
Illustrates hand-operated screens installed at the Dundee Falls, Me., plant of the Presumpscott Electric Co.

1916—Morrison, C. E., and Brodie, O. L. Masonry Dam Design. 1916. (John Wiley Sons, N. Y., 267 pp.)
Pages 16-19 discuss the question of ice thrust in reservoirs and dams using specific examples. Pages 115-119 work out, according to the formulas, the proper design for dams, allowing for ice pressure.

1917—Creager, W. P. Engineering for Masonry Dams. 1916. (John Wiley Sons, N. Y., 229 pp.)
Ice pressure is discussed, pp. 37-39, including the subject of ice thrust and allowances which must be made when designing a structure. A table gives ice pressures adopted for various dams; the design of reservoirs. Pages 78-104 include examples showing the necessary allowances to be made in building to account for ice pressure.

——**Preventing Ice Troubles at a Connecticut Hydro-Electric Plant.** 1917. (In *Electrical World*, v. 69, pp. 184-185.)
Illustrates the arrangements constructed for disposing of ice that clogs the forebay.

——**Smith, J. C.** The Hydro-Electric Power Plant of the Cedars Rapids Manufacturing & Power Co., at Cedars, P. Q. 1917. (In *Canadian Engineer*, v. 32, pp. 211-213; Abstract from *Transactions*, Canadian Soc. Civ. Engrs., article not available.)
A sketch is given to show the general plan and layout of the embankment. A description is given of the power house and the main units, as well as of the method used in diverting ice from the intake.

1918—Daily, C. M. Keeping Ice from Ports of Water Intake. 1918. (In *Engineering News-Record*, v. 80, pp. 900-901.)
Describes the plan adopted by the St. Louis Water-Works authorities to protect intake ports. The device illustrated consists of a timber boom with buoyancy cylinders.

——**Davis, A.** Protection of Keokuk Dam Gates from Ice Pressure by the Use of Air. 1918. (In Stone & Webster *Journal*, v. 22, pp. 199-206.)
A history of the ice action at Keokuk, together with some conjectures as to its causes, followed by a description of a very successful air system for dealing with ice blocking.

——**Groat, B. F.** Ice Diversion, Hydraulic Models, and Hydraulic Similarity. 1918. (In *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), pp. 1138-1151, and discusssion.)
An illustrated paper which treats of a new method of diverting surface water and all floating materials carried thereby, for the purpose of preventing jams in canals and rivers.

——**Installing Apparatus to Protect Dam from Heavy Ice.** 1918. (In *Electrical World*, v. 71, p. 778.)
Describes the use of a 5-h.p. air compressor to prevent ice formation on spillways and gates of dams and hydro-electric plants.

——**New Installation Keeps Ice Away from Keokuk Dam.** 1918. (In *Engineering News-Record*, v. 80, pp. 821-823.)
A pipe line across the Mississippi River supplies a constant current of air up stream and prevents the formation of ice on the gates.

——**Pierce, C. H.** Effects of Ice on River Discharge. 1918. (In *Journal*, Boston Soc. C. E., v. 5, pp. 414-419.)
Stream flow for the canal and weir types of waterways is computed, theoretically. Using these calculations as a basis the effects of frazil, surface, and anchor ice cover are worked out. Graphical results show the effect of ice on river discharge.

——**Purcell, J. M.** Ice in Hydro-Electric Plants. 1918. (In *Power*, v. 48, p. 951.)

——**Slush Ice Prevention at the Denver, Colorado, Water-Works Intake.** 1918. (In *Engineering News-Record*, v. 80, pp. 615-616.)
A sheet-piling channel in the forebay and the maintenance of a steady flow of water through a 60-in. conduit prevents the formation of ice around the intake screens.

1919—Bromley, C. H. Operation at Holtwood. 1919. (In *Power*, v. 49, pp. 850-854.)
Describes the manipulation of screens and trash racks to prevent ice clogging at turbine intake. The racks are made in sections, one or two sections being removed. This article discusses at some length the difficulties encountered with débris and how the condition was treated.

——**Russell, S. R.** Method of Breaking Up Ice Gorges with Dynamite. 1919. (In *Engineering and Contracting*, v. 51, pp. 217-218.)
Describes and illustrates three ways of placing dynamite charges for varying types and positions of ice jam.

——**Stickle, H. W.** Monongahela River Navigation. 1919. (In *Proceedings*, Engrs. Soc. W. Pa., 34, pp. 245-266; Discussion, pp. 267-277; Abstract, *Engineering and Contracting*, v. 50, pp. 283-284.)
Pages 252-253 describe methods used in dynamiting ice blocks in the vicinity of Lock No. 5.

——**Wilson, R. M.** Design of Hydro-Electric Plants for Combatting Ice Troubles. 1919. (In *Journal*, Eng. Inst. of Canada, v. 2, pp. 383-395; Discussion in *Canadian Engineer*, v. 36, pp. 237-242.)
An article showing how ice troubles at turbine intakes have been eliminated by the Cedar Rapids Manufacturing & Power Co. A special type of trash rack is described, which is built in sections, allowing a portion to be removed. A considerable amount of railroad ties, stumps, stones, and other débris was found in the runners, but this condition was satisfactorily met. Discussion follows the article.

1920—Balderston, C. Ice Inside Cast-Iron Column Raises Floor. 1920. (In *Engineering News-Record*, v. 84, p. 1166.)
Relates a case of ice formation inside a cast-iron column which resulted in lifting the column several inches off its base and pushing a heavy granite sidewalk a like distance.

——**Carpenter, J. S.** Ice Protection for Hydro-Electric Plant. 1920. (In *Power Plant Engineering*, v. 24, pp. 1125-1126.)
Explains methods for deflecting ice jams. Ice forms around speed rings of turbine, causing water passages to become clogged.

——**Concrete Road Uninjured by Weight of Ice Jam.** 1920. (In *Engineering News*, v. 85, pp. 29-31.)
Gives example of Clark's Ferry at junction of Juanita River with Susquehanna, where a concrete pavement remained undamaged after a heavy ice jam.

——**Eckhard, G. F.** Large Highway Bridge Wrecked by Pressure of Cake Ice. 1920. (In *Engineering News*, v. 84, pp. 902-904.)
Gives example of the Brattleboro (Vt.) steel span of 330 ft. pushed off its supports by floating ice. Weight of steel in bridge was 270 tons. Distortion of floor system increased until horizontal deflection of the lower lateral system became at least 2 ft.

——**Groat, B. F.** Ice Diversion for St. Lawrence River Power Co. 1920. (In *Canadian Engineer*, v. 39, pp. 545-552; Abstract in *Engineering and Contracting*, v. 55, pp. 45-46.)
Discusses construction of submerged weir and ice diversion works in South Sault Channel of St. Lawrence River. Ice jams cause high back-water, thus reducing the head on the turbines during such periods, sometimes by as much as 10 ft.

——**Merriman, M.** Ice Pressure. 1920. (In Am. Civil Engr's. Handbook, 4th Edition.)
Example given of a sheet of ice 150 ft. in width tipping a masonry bridge pier weighing 1 000 tons, 2 in. out of plumb (p. 435). Pressures due to ice have been found to be 30 000 lb. per sq. ft. for field of melting ice 1 ft. thick, striking a pier and crushing its way past (p. 709).

1921—Murphy, J. Ice Formation and Its Prevention. 1921. (In *Canadian Engineer*, v. 40, pp. 291-292, 294.)
A review of the progress of investigations into methods for the prevention of the formation of ice in hydro-electric practice. States that it is possible to prevent ice from adhering to valves, gates racks, penstocks, turbine runners, and casings by the proper use of heat.

1922—Gaby, F. A. Queenston-Chippewa Development of the Hydro-Electric Power Commission of Ontario. 1922. (In *Journal*, Am. Inst. Elec. Engrs., v. 41, pp. 508-526.)
Discusses the design of the intake, taking into account the floating and frazil ice which forms on the Niagara and Welland Rivers. A submerged intake consisting of gathering tubes and a surface intake forming a concrete barrier to ice, are part of the layout which is proving successful.

——**Ice Handling Methods.** 1922. (In *Proceedings*, Nat. Elec. Light Assoc., v. 77, pp. 526-529; Abstract, *Power*, v. 55, pp. 948-949.)
The type of intake rack, designed in two parts and ready for quick removal in case of sudden frazil ice, used at the Cedars Rapids hydro-electric plant in Quebec, is briefly described, but well illustrated on p. 528.

——**Stevens, J. C.** Winter Overflow from Ice Gorging on Shallow Streams. 1922. (In *Transactions*, Am. Soc. C. E., v. LXXXV (1922), pp. 677-696; Discussion, pp. 697-698.)
The facts gathered during the author's study of the winter overflow of the Madison River, Montana, are used as the basis of his paper on the occurrence, in the coldest part of the winter, of frazil ice in streams too turbulent to permit surface ice formation. The effects of this ice on bridges, piers, reservoirs, and channels are described, together with remedial measures.

——**Wegmann, E.** Design and Construction of Dams. Seventh Edition. 1922. (Wiley, N. Y., 555 pp.)
Quaker Bridge Dam is designed to resist an ice thrust of 43 000 lb. per lin. ft. (p. 160). Wachusett Dam will resist an ice pressure of 23.5 tons per lin. ft. (p. 194). Cross River Dam designed to resist ice pressure of 24 000 lb. per lin. ft.; Croton Falls Dam, of 30 000 lb. (pp. 203-204).

1923—Anderson, C. N. Ice Prevention on Trash Racks by Electric Heating. 1923. (In *Electrical World*, v. 82, p. 340.)
Illustrates racks adapted for electric heating at the Kykkelsrud Power Station, Norway.

——**Skerrett, R. G.** Reducing Ice Pressures with Compressed Air. 1923. (In *Canadian Engineer*, v. 45, pp. 181-185.)
Describes subaqueous jets of compressed air utilized to prevent ice formation on gates of the Mississippi River Power Co.'s plant at Keokuk, Iowa.

——**Walker, W. T.** Ice Problems of Hydro Plants. 1923. (In *Engineering and Contracting*, v. 60, pp. 1254-1264; Abstract of paper before American Society of Civil Engineers, 1923.)
Suggests methods of relieving gates of ice pressure; shows the importance and necessity of studying the effect of frazil, anchor, and shore ice on piers, canals, and hydro-electric structures.

1924—Bombing an Ice Jam. 1924. (In *Aviation*, v. 16, p. 459.)
Reports on the use of planes in dropping bombs on an ice jam in the Platte River, between Columbus and Valley, Nebr.

——**Effects of Ice on River Flow at Keokuk.** 1924. (In *Electrical World*, v. 83, p. 336.)
It is concluded that the flow during any winter will depend on the flow at the time of the first freeze-up. This initial freeze-up will reduce flow to 40% in 6 days and keep it at that for 5 days, after which 75 to 80% is gradually reached. Reasons given.

——**Freeman, John R.** General Review of Current Practice in Water Power Production in America. 1924. (In *Transactions*, First World Power Conference, v. 2, pp. 359-394.)
Discusses ice troubles and their cure (pp. 368-370).

——**Hogg, T. H.** Ice Problems at Hydro-Electric Plants. 1924. (In *Canadian Engineer*, v. 47, pp. 498-500.)
Frazil ice discussed, followed by a description of racks, ice skimmers, and immersion heaters. The specific ice problems of the St. Lawrence District are discussed.

——**Hollow Concrete Bridge Piers Built Like Chimneys.** 1924. (In *Engineering News-Record*, v. 93, p. 757; Discussion, v. 93, p. 924, and v. 94, p. 164.)
Ice pressure on piers of the Kentucky Hydro-Electric Co.'s Kennedy Bridge.

——**Hyatt, R. S.** Preventing Ice Formation on Water Turbines. 1924. (In *Electrical World*, v. 83, p. 386.)
Illustrates a successful arrangement of turbine parts, showing location of heating elements.

——**Kennedy, J. H.** Ice Control Methods at Shelburne Falls. 1924. (In *Power Plant Engineering*, v. 28, pp. 895-896.)
Describes the work which must constantly be carried on to keep the channels of the New England Power Co.'s Deerfield River plant in operation. Ice is held on either side of the channel in solid blocks by its attachment to booms. In this way the ice is melted slowly in the spring and a great deal of damage prevented.

——**Ruths, A.** Ice Troubles in Norwegian Water Power Plants. 1924. (In *Transactions*, First World Power Conference, v. 2, pp. 771-788; Abstract, *Engineering*, v. 118, pp. 73-74; also *Engineering and Contracting*, v. 62, pp. 805-809.)
A complete discussion of the nature of formation and relative danger of frazil, anchor, and sheet ice. Conditions existing at the Vamma Hydro-Electric Plant in Norway are described, together with means used to remove and prevent ice on turbines, gates, and at intakes.

——**Samsioe, A. F.** Measures Taken in Sweden Against Ice Troubles at Water Power Plants. 1924. (In *Transactions*, First World Power Conference, v. 2, pp. 806-811.)
Discusses the damage done by ice jams and ice sludge. Gives some figures on the amount of pressure exerted on dams by ice jams.

——**Westerlind, A.** Recent Practice in Swedish Water Power Engineering. 1924. (In *Transactions*, First World Power Conference, v. 2, pp. 338-358.)
Discusses the allowances made for ice pressure in designing dams, piers, and hydro-electric plants (pp. 341-344).

1925—Broadhead, A. P. Prevention of Damage from Anchor Ice. 1925. (In *Electrical World*, v. 86, p. 1108; also, *Engineering and Contracting*, v. 65, p. 53.)
Discusses the nature of formation and danger from such ice.

——**How to Blast Ice Jams.** 1925. (In *Engineering and Contracting*, v. 63, pp. 489-490.)
A 30 to 40% grade of gelatine dynamite is recommended for breaking ice on rivers. Directions are given for placing and firing the charge.

——**Ice Shove on High Bridge Piers.** 1925. (In *Engineering News-Record*, v. 93, p. 924; v. 94, pp. 164-165.)
L. F. Harza gives the resisting moment against overturning of the 22-ft. piers of the Kentucky Hydro-Electric Co.'s Kennedy Bridge, at 49 600 000 ft-lb. in a direction at right angles to the center line of the bridge and 30 800 000 ft-lb. in the direction of the bridge.

——**Loughland, G. E.** Ice Formations in the Main Sierra Canal. 1925. (In *Engineering and Contracting*, v. 63, pp. 775-780; copied from *Military Engineer*.)
Defines the different types of ice, conditions of formation, and methods of combatting them, in California.

——**Lowy, H. W.** Overcoming Ice Difficulties at the Holtwood Power Plant. 1925. (In *Journal*, Boston Soc. C. E., v. 12, pp. 142-149.)
Describes and illustrates the effects of river, forebay, and frazil ice on the operation of the Pennsylvania Water & Power Co.'s plant on the Susquehanna River. Statistics are given to show the power reduction due to frazil ice during 1919-1924.

1926—Barnes, H. T. Ice Conditions in St. Lawrence River. 1926. (In *Canadian Engineer*, v. 50, pp. 207-208.)
Discusses the effect of ice on water levels, the advantages provided by Nature, and describes a method worked out for the conservation of heat of Lake Ontario for ice prevention.

——**Carroll, E.** Guniting Up-Stream Face of Dam as Precaution Against Ice Damage. 1926. (In *Engineering News-Record*, v. 97, p. 432.)
Illustrates the application of gunite to protect Basin Creek Dam, a monolithic structure impounding the water supply of Butte, Mont. Surface ice on the reservoir caused severe damage to the dam face.

——**Dispositif d'Evacuation des Glaces et Corps Flottants** dans les Usines à Turbines Verticales. (Removal of Ice and Trash in Vertical Turbine Installations.) 1926. (In *Génie Civil*, v. 88, p. 237; also, *Power*, v. 63, p. 977.)
Illustrates a trash rack installed behind a vertical turbine to prevent débris from entering the turbine runner.

——**Franz, J.** Preventing Ice on Flash-Boards. 1926. (In *Electrical World*, v. 88, p. 1175.)
The Rockdale Hydro-Electric Station of the Southern Berkshire Power & Electric Co. has installed a galvanized iron pipe device connected with an air compressor which agitates the water and prevents freezing.

——**Ice Gorge Puts Power Plants Out of Commission.** 1926. (In *Power*, v. 63, p. 391.)
Announces the use of "thermite", a chemical developing a heat of 5 000° Fahr., in breaking up ice at Oil City, Pa.

——**Ice Jams Do Much Damage Along Allegheny River.** 1926. (In *Engineering News-Record*, v. 96, p. 503.)
Brief description of the destruction of three spans of the Big Rock Bridge, below Franklin, Pa., due to the breaking up of a severe ice jam.

——**Reid, C. R.** Overcoming Slush Ice Troubles. 1926. (In *Power*, v. 63, p. 977.)
Suggests means of preventing such ice entering the intake.

——**Removal of Ice and Trash in Vertical Turbine Installations.** 1926. (In *Power*, v. 63, p. 705.)
Gives diagram of proposed method of passing ice and trash through water power plant.

1927—Schwenkhagen, H. Bisherige Ergebnisse der Eislastforschung der Studiengesellschaft für Hochspannungsanlagen e. V. (Previous Results of Ice Pressure Research of the Studiengesellschaft for High-Tension Installation.) 1927. (In *Elektrizitätswirtschaft*, v. 26, pp. 10-18.)
Gives results of research on the formation and effect of ice on high-tension electric transmission lines.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

REPORT OF THE COMMITTEE OF THE IRRIGATION DIVISION ON "A NATIONAL RECLAMATION POLICY"

Discussion*

BY MESSRS. F. N. CRONHOLM, R. I. MEEKER, AND E. B. DEBLER.

F. N. CRONHOLM,† M. AM. SOC. C. E. (by letter).‡—No matter how business-like and favored a new reclamation policy might prove as to return of capital invested in irrigation works, the fact remains that settlers who chance to acquire good land which, because of its physical conditions, can be quickly and economically farmed, will prosper; and the settlers not so fortunate will fail. As long as the study of the problem is narrowed or confined so that it does not go beyond bringing water to each farm unit, there will be little, if any, improvement. Probably no one knows the relative cost per acre of making lands ready to seed as against that of bringing water to the farm. It is reasonable to assume that the cost is more per acre to bring a farm to the same plane of perfection as the irrigation works, and if this is approximately the case, the writer feels that the settlers' viewpoint on the problem has not been given as full consideration as it should have. Certainly, the magnitude of this phase of reclamation would seem to warrant greater analysis.

The writer believes that more should have been done as regards Government reclamation projects than merely bringing water to the highest part of a farm unit. If it is reasonable to have the works built by the Government as efficient, as lasting, and as dependable as they are, then it seems reasonable that the lands served be as well provided. Truly, they are inseparable; so if there is justification for excellent dams, diversion structures, canals, and laterals ready to serve, then there is like justification for the canaled area leveled and, in every particular, ready to farm.

What the U. S. Bureau of Reclamation needs, if it does not already possess it, is a policy enough ahead of the times so that when a project is completed everything about it will be attractive. An irrigation project is, after

* Discussion of the report of the Committee of the Irrigation Division on A National Reclamation Policy, continued from February, 1930, *Proceedings*.

† Los Angeles, Calif.

‡ Received by the Secretary, January 18, 1930.

all, although indirectly, a kind of plant intended to produce food, and once it is commenced it is not complete until the land is made ready to farm. Directly from the viewpoint of the State, as well as that of the Federal Government, the main purpose of a project undertaking should be the building of a community of happy, wholesome people.

There are vast areas of desert already peopled to the benefit of both State and Government, and such work by all means should continue. There are also vast areas elsewhere, long since settled, that are dying or that are already dead. It should be the business of the Bureau of Reclamation to plant an example, as it were, here and there throughout such localities and revive crop output by working the land to capacity, making the community idea of settlement so attractive that people will abandon other places that are hopelessly isolated and impossible to live in happily or to farm profitably. The cost of such community building should be pro-rated among those benefited, namely, the settler, the State, and the Federal Government. Make the sum to be refunded by the settler such that he can pay both the principal and interest and always feel decently cheerful about his contract as compared with what he might have done elsewhere.

What about the vacant areas of existing irrigation projects or extensions? Before they can be disposed of must these lands not be made attractive? Truly, there is more involved than the reasonableness of the annual maintenance and the repayment of construction charges. The prospective farmer will estimate, as best he can, to what extent he will be obliged to apply what monies he has toward getting the land ready to produce. Whatever the outcome, there is no sound logic in permitting "Tom, Dick and Harry" to hazard monies invested by the Government by allowing them to level land, even at their own expense. The preparation of the surface soils for the application of water as done by settlers has, in many cases, been against best crop output, against economic application and use of water, and has been a permanent detriment compared to what would have been the case had leveling been rightly done in the first place.

For instance, consider the case of a farm unit on which the surface soils happen to be more productive than those of greater depth. These rich soils are likely to be wasted by finding lodgment in hollows, whereas they should be saved to spread over the surface as the farm unit is leveled. However, the settler is much like any other person. He is apt to overlook many things; in fact, he may be obliged to do so because of the necessity of producing something before he "goes broke". Time is a factor. He will eat up what he has before harvest time if he does not hasten planting. Therefore, saving top-soil becomes secondary as does the methodical preparation of the surface. The settler plants first probably an already level patch, irregular in contour. Water is brought to this naturally level piece of land by short-cutting across the farm unit—meandering around this hollow and that bump. He has now permanently disfigured, as well as destroyed, his unit for future maximum economic crop output.

Water is brought to land for a purpose, and it is the purpose in which the entire nation, as a people, is primarily interested. Therefore, while water is

indispensable in arid regions, it is but a means to an end, and this objective should be the same in Alabama as it is in Idaho. In the beginning the purpose of the green spots to be created by the U. S. Bureau of Reclamation in the deserts of the Western States was, it is stated, for food production in given areas, and at the time this was assumed as ample reason for expending monies for irrigation works; possibly so. At any rate, it brought about a beginning by creating a department in the Federal Government that should continue in the good it is doing and can do. The objective of food production of long ago must be enlarged upon, however, or changed, in order that it may be more nationally adapted. Make this purpose as applicable to Alabama as to Idaho. Let the objective be community building, reviving of soils, and promoting the interests of the settler. Do work in every State for the purpose of bringing up the average. Make irrigation works, stump pulling, land leveling, draining of swamps, or adding fertility to soils secondary in so far as they are but a means of bringing about the objective. Preparing lands to the point where they are ready to farm and offering them to people at a price and on terms that are attractive and well within their ability to pay is an undertaking that—coupled with assistance and guidance needed in the matter of suited crops, market outlets, etc.—would provide a greater certainty of return of investment to the Government than when the Government undertakes to do but one of the major necessities, such as bringing water to land. The program should include any assistance or guidance needed in the matter of choosing suitable crops, market outlets, etc.

Converting rough, raw land into producing farms has been done by settlers poorly fitted, both as to knowledge and money. Poor and humble as many of them were, each one must have believed that his progress would be forward. As a matter of fact, many people lost their entire savings and quite permanently wrecked themselves. Every one having a part in this first work was forced to a lower standard of living for quite a time and, as a result, no one of them is inclined to bestow gratitude. If it costs from $60 to $120 per acre to bring water to a farm, it costs on an average about as much again to prepare land ready to farm, that is, with the understanding that the work is done as lavishly and completely in one case as in the other. Therefore, it seems that the settler, in preparing his land to farm, even though inadequately, has certainly done his part, whether it was on a Federal or a private enterprise.

To expect private enterprises to solve the problems of Federal Government undertakings is of no use. Policies and methods of merit, differing from those applied by the Government, have always been available and would, undoubtedly, have been adopted forthwith in the beginning had such policies and methods been most adaptable. The writer has observed that the Government has always been progressive. The men responsible for making decisions since the infancy of the Reclamation Service are to be commended, and, considering the magnitude of the work accomplished, have an enviable reputation in that they made so few mistakes. The Government's most questionable projects are poorly located from the point of view of marketing and climate. The idea that the more private a project is, the more successful it will be, or

the more public it is, the more hazardous it will be, should not be taken too seriously. Take Imperial Valley as an example. This project is what might be termed "half and half" since it is not entirely Government and not entirely private. It has ample area so as to be representative; it has many sunshine hours, exceptional growing climate, and fair marketing conditions. At that, it has seen many "ups and downs" and most certainly, it is not yet "out of the woods". Comparisons are to no end and, as previously stated, of no value.

What irrigation projects lacked at the time the Government began its work was scope. Irrigation practice, outside of possible district projects, was strictly commercial, first, last, and all the time. Works were built with total disregard to future upkeep, and it was the settler who shouldered the burden of paying the cost as well as a profit to the promoters. Salesmanship was a main feature because good and lasting works could be slighted or needed features could be eliminated altogether, depending on the amount of sales talent. As a consequence, people settled the areas more out of the hope and glowing promises engendered by high-powered sales talk than because of any material thing of worth. Re-sales resulted, and the next settler was able to benefit somewhat by the labors of the man he succeeded. While this may be irrelevant and immaterial and of no moment, the writer believes that if private and district projects had gone another step toward making lands ready to farm and had been leveling land when the Government entered the field, the Government would have adopted such a policy also and, to-day, all projects would be better off financially and their settlers more prosperous.

Perhaps the most serious lack was the foregoing, namely, the absence of land leveling as a practice in project construction. Besides there was the mistake of using too generously, in estimating acre costs, the lands within the reach of the projects' distribution systems. Many of these lands, on detailed examination, would have been excluded. The suitability of the rougher lands, from the viewpoint of the prospective settler and his ability to pay, was not studied as completely as it should have been. Fitness of lands to farm should have been given the same study as the building of irrigation works.

It is all right in 1929 to say that twenty-five years ago project settlers should have been only those with $5 000, or more. That was considerable money in 1904, and people who moved about in covered wagons were, for the most part, hard-working folk glad of employment in the building of project works. Often it was these same people who, later, out of hard-earned savings, became project settlers. Such people are gradually disappearing like the buffalo and the antelope.

The only solution to land settlement projects, whether or not the question of water is involved, or whether the work is done by the Federal Government, or by private or district organizations, is to do more toward making lands ready to farm. Even to-day people with $5 000 or $10 000 are likely to wear too good clothes for farming. What, in fact, is there against helping, or rather, creating opportunity for, the man with $2 000, or less? There are many such worthy people, and, generally speaking, if the Government is interested

in social uplift and community building through giving impetus to crop output and organizing marketing, then it is these people who have the greatest need.

If there is ever a time when a man needs all his allotted land in shape to farm, it is in the beginning. For instance, if in a given locality it will require, ultimately, say, 80 acres to support a family in an American way of living, then it is imperative that the entire unit be ready to seed as soon after settlement as growing seasons will permit. The size of a farm unit should, in any case, be sufficient to support the average family properly, and allow for growth and education. Decent, respectable living with forward progress contingent on honest effort can rightfully be expected by any man willing to work to improve his condition, and under no circumstances should the Government lend itself to an undertaking that would hazard such progress. Most certainly it did this during the early project settlement and, in fact, does so now every time it permits a settler on raw land not ready to seed. Such risking of a man's chance is diametrically opposed to what the Government's purpose should be.

In spite of the foregoing, the U. S. Bureau of Reclamation ranks high in accomplishment. There is no organization in the nation that is cleaner, or that can show more good per dollar invested. The Service must become more human and more National in scope. It must do reclamation work that will draw people from various States to a given locality for a specific purpose and thus create new life and bring hope nearer realization. At present, the problem is, what to do with unsettled parts of existing projects so as to hasten their settlement. The writer suggests the following as a beginning:

1.—Choose a project having unsettled land that is so situated geographically that additional crop output could be profitably marketed.

2.—Furnish maps showing the topography of each farm unit in the rough and others showing how the same land will look when it is leveled, ditched, and diked, and the necessary control structures are in. Show these maps to the prospective settler in order that he may see what is before him as well as choose what particular unit of those available he likes best (allowing progressive settlement only).

3.—Level, not part of the farm, but all of it; and do this following the most approved method according to soils, in a manner allowing for a reasonable range of crops. Lay out the work so that the farmer may manage without hired help if possible. Keep in mind economy of water for the good of crops.

4.—Besides leveling the farm, turn the water on to it; if, in wetting, any areas settle, re-level that area. Make sure the entire unit is ready to plant. Furthermore, provide outlet for surplus irrigation water and, in the farm layout, give some thought to possible future ground-water relief should that become necessary.

5.—Do not prepare lands to seed until the area has been applied for by a qualified settler who shall be required to put all the land in crop beginning with the first growing season and keep it in crop for two full seasons before he is allowed to file on the farm unit.

6.—Have each prospective settler acquire the necessary credentials to become a conditional settler from a local board entrusted by the people of the project with determining an applicant's fitness, the board not being limited by a prescribed minimum money requirement.

7.—Collect in advance project maintenance charges. Add the cost of leveling and otherwise preparing the farm unit to project costs against the land, and of the total take a fair amount to be amortized in a reasonable time by the settler. Have him make the first payment and interest on deferred payments prior to the third season and yearly thereafter without fail. Make the amount to be ultimately repaid the Government of such size that the farmer will not fail because of it, but, at the same time, make it an obligation generally conceded as ample.

8.—Do not charge for water used during the first two seasons. If the prospective settler pays all other expenses involved in cropping the entire unit twice, and makes ready to begin paying off the project debt by the third year, he will be doing well. After all, the would-be farmer will compare every location available to him, weighing advantages against disadvantages, before finally deciding on a particular project and what it has to offer. Once a man has become involved in a proposition, it is well that he continues to be satisfied.

9.—Build no project without the prior approval of the State, or States, within which it is located, and then only on condition that the State, or States, be interested enough to guarantee the Federal Government the full amount of estimated costs in case of the settler's failure to meet the terms of his contract.

The foregoing list resolves itself into a plea for "ready-to-farm lands" and would involve a great deal in the matter of organization, although, presumably, the educational work needed for its authorization and the procuring of needed monies are the more difficult of accomplishment. It is acknowledged that something must be done to settle lands on Government projects now vacant. Personally, the writer cannot help feeling that preparing farm units ready for seeding would bring sufficient settlers of the right kind to these areas.

In further clarification of Suggestion No. 7, it seems possible and equitable to the farmer as well as to the Government to fix a rate of interest on deferred payments that will, in the end, make up the amount written off in the beginning so that, as a matter of fact, the Government will be repaid in full. In this way the Government would not be a money lender charging rental, as it were, for its money, but the farmer would be paying interest nevertheless. For instance, say the project costs $150. The Government would only charge the farmer $75, but it would charge him interest at a rate which, in a certain term of years, will ultimately repay the total project cost to the Government.

If something along the lines suggested could be authorized, the writer feels sure that it would succeed. Making lands ready for seeding should be done for the Southern States as an experiment, and with the two demonstrations, one in the West and one of fair acreage in the South, the writer believes that an adaptation of the U. S. Bureau of Reclamation to greater National good could be attained, and, in turn, the objectional political phase lessened. The good the Service would then be doing would be more broadly understood and welcomed.

R. I. MEEKER,* M. AM. SOC. C. E. (by letter).†—The National Reclamation Act of 1902 was framed with the purpose of using monies from the sale of the public domain in the arid and semi-arid West for the reclamation of other arid lands. Under the Oil Leasing Act of 1920, 52½% of the income from oil and minerals from public lands was added to the National Reclamation Fund, and under the Federal Water Power Act of 1920, 50% of income from that source was also added to the Fund. Federal reclamation to date, therefore, can be stamped as Western aid to Western development since Western resources have been used to finance irrigation development.

Income from public lands in the arid States is dwindling. Future irrigation projects as outlined by surveys will be large and costly and with a much greater unit cost per acre than during the past twenty years. Many proponents of reclamation are advocating greater Federal participation. The supplemental report of the Committee‡ mentions the need of National aid to reclamation by direct appropriations from the Federal coffers for public works for irrigation—this in addition to the present Reclamation Fund. The precedents of Federal aid for navigable rivers, highways, aviation, flood control, and inland waterways are cited as the arguments for such National aid for future water utilization.

If such procedure is imminent, and it so appears, then a new phase of financing arid reclamation is in the offing, and Federal reclamation will be more than a term; it will be a fact, and the relations of Federal and State activities in water matters will assume a matter of extreme importance to the Western States in view of past experience with Federal reclamation.

The desire for "Federal aid at any price" (which is so rampant in recent years), and at the price of local government, is not believed to be advantageous to the West.

Federal and State relations and co-operation in water matters are specifically referred to in Items (3) and (7) of the Committee's report,§ and one page of the supporting statement is devoted to discussion thereof.‖ The writer desires to direct attention to an unenviable feature of past reclamation and water utilization with the hope that future Western reclamation may be devoid of a repetition of such offenses, especially if National funds are to be the backbone of such development.

The ownership and control of Western rivers lie in the Western States as announced in constitutional provisions and enabling acts of the various States, and as sustained in the Kansas *vs.* Colorado, U. S. Supreme Court decision. Paragraph 8 of the Federal Reclamation Act specifically states that Federal reclamation shall conform to and be subservient to State water laws. The history of National reclamation does not chronicle complete acquiescence to State laws in such matters.

The National Reclamation Act was scarcely passed before the U. S. Reclamation Service intervened in the U. S. Supreme Court water suit between

* Cons. Engr., Denver, Colo.
† Received by the Secretary, January 24, 1930.
‡ *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1198.
§ *Loc. cit.*, September, 1928, Papers and Discussions, p. 2099.
‖ *Loc. cit.*, May, 1929, Papers and Discussions, p. 1198.

Kansas and Colorado concerning the Arkansas River, claiming National title to and right of control of the waters in non-navigable streams of the Western States. Furthermore, the Reclamation Service asked the Court to announce a doctrine of priority in water matters regardless of State lines and irrespective of whether the riparian or appropriative doctrine obtained in adjacent States. The Supreme Court held adversely to the Reclamation Service and confirmed the rights of the States as to complete jurisdiction over their rivers.

The entry of the Federal Government on river systems of the West immediately chronicled the commencement of Federal interference in State water matters and opposition to proposed private power and irrigation projects even where the rights of such projects in many instances were anterior to the rights of Government projects. Illegal and injurious embargoes by the Federal Government were imposed on some of the river systems for the so-called protection of Government projects. Canal and reservoir rights of way were denied by the Department of the Interior at the instigation of the Reclamation Service on the grounds that the construction of such projects would jeopardize the water supplies of the Federal projects, despite the protection under State laws of their rights by the doctrine of priority. Not satisfied with priority dates of Government projects, the Reclamation Service blocked and "killed" power and irrigation projects, especially reservoir features thereof, within river systems where Government projects were located. Trans-mountain diversions through the Continental Divide were opposed and defeated by denials of rights of way by the Department of the Interior on the flimsy ground of injury to the Yuma Project on the Lower Colorado River, etc. Hence, while the Reclamation Service and affiliated Federal bureaus had no direct control over Western rivers, they maintained indirect control of parts of river basins and water supplies through control of Government land, giving preferential status to Federal projects.

International disputes were unfairly invoked to the detriment of the development in river basins in the United States. A list of private projects blocked and "killed," with the means resorted to in "killing" them, would not grace the present Federal accomplishments.

Even at this late date Federal reclamation officials are claiming ownership of return flow and drainage water from Federal projects with sales of the water to other projects lower down on river systems. The U. S. District Courts have become a refuge for Federal projects in return-flow disputes, and a line of U. S. District Court decisions is being forged favorable to Government ownership of return-flow waters.

Federal bureaucracy has left an unnecessary trail of blighted hopes, blocked and "dead" power and irrigation projects, now submerged in the rainbow of accomplishment. Section 6 of the National Reclamation Act specifies that ultimately the management and operation of Federal irrigation works shall pass to the water users served thereunder. This local management is gradually coming to pass, but all too slowly. Early local management and operation of Federal projects will go a long way toward the elimination of Federal interference in water administration. The assumed preferential status of

Federal projects in water matters will then collide with State laws, and it is hoped that such assumed preferential status will be effaced.

Item (15)* of the Committee report, *in re* turning of reclamation projects over to local management at an early date is highly desirable in order to get away from a part of Federal interference and opposition to State water laws and river administration.

The writer realizes that the encroachments of Federal reclamation on State water control and the special privileges assumed by Federal projects have been made possible partly as a result of local encouragement by politicians and water users under some of the projects, and even State legislators who were willing to go the limit in getting Federal monies expended on their river systems. This situation still persists as to proposed Federal projects. The expenditure of large sums of money from the National Treasury on reclamation and water utilization in the arid West should not be looked upon with favor unless State and Federal relations are clearly defined, and abuses of the past are avoided.

The August, 1929, proposal of President Hoover authorizing the transfer of the remaining public lands to the Western States, and voicing a hope that a line of action might be followed that would "check the growth of Federal interference in affairs of essentially local interest, and thereby increase the opportunity of the States to govern themselves," is a consummation devoutly to be wished in water matters of the arid West.

The writer heartily concurs in Item (8)† of the Committee report to the effect that future stream regulation shall give preference to supplemental water supplies for existing Federal or private projects before new projects are considered.

E. B. DEBLER,‡ M. AM. SOC. C. E. (by letter).§—The Committee proposals,‖ briefly stated, comprise:

(*a*) Stream development in accordance with general plans formulated by the Federal and State Governments with basic data obtained by them.

(*b*) Change in Federal aid from that of waiving of interest to the assumption of part of the cost with low rate of interest to be charged on the remainder.

(*c*) Finding of feasibility and recommendation by experts on all phases to be a prerequisite to the approval of new projects for construction, with the States to share the responsibility for the selection of projects; the elimination of State lines and local interest in such selection.

(*d*) Construction preference to be extended to projects providing supplemental water supplies, and the reclamation of new lands to be delayed.

(*e*) Construction repayments to commence promptly upon the completion of each unit of the project, to be initially light, and to extend over a long term, with forfeiture of land upon delinquency.

(*f*) Aid in settlement to be provided by States and local interests.

* *Proceedings*, Am. Soc. C. E., September, 1928, Papers and Discussions, p. 2100.
† *Loc. cit.*, p. 2099.
‡ Hydr. Engr., Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo.
§ Received by the Secretary, February 6, 1930.
‖ *Proceedings*, Am. Soc. C. E., September, 1928, Papers and Discussions, p. 2098.

The lack of an official discussion by the Committee, as a whole, of the annunciated principles leaves a wide latitude of interpretation for the reader. The writer has necessarily relied largely on Mr. Lippincott's discussion* for guidance in this situation.

The desirability of planned stream development needs no defense; but acceptance thereof by States will not always be readily obtainable, as witness the wrangling over the Boulder Canyon (Colorado River) and the St. Lawrence River developments.

Change in Federal Aid.—The proposed change in form of subsidy from a waiver of interest on construction cost to one of outright donation of a part of the necessary works, provided the remainder of the works are constructed by other agencies, has one very decided advantage over present conditions. It obviates the always unpleasant task of collections which, in the nature of things, is always difficult when anything is bought under the instalment plan, whether it be a radio or a water right. Mr. Lippincott at one place advocates that Federal aid take the form of reservoir construction on navigable and interstate streams, with similar aid on non-navigable streams to be provided by the States. Such a course would discriminate heavily against the inter-mountain States having no navigable waters. On the other hand, States fortunate enough to be possessed of navigable waters would receive a double benefit of subsidized water supply and subsidized transportation. If such a plan were enacted, navigation on mountain streams might suddenly be found to have surprising possibilities. Mr. Lippincott appears to recognize the exceptions that would be necessary in that he considers it only fair that a Federal appropriation be made for the construction of the All-American Canal. While storage construction would go far toward solving Sacramento Valley problems, it would be only an insignificant aid to the Columbia Basin Project. In justice to the Committee as a whole it must be noted that the report is not specific on the development features to be donated by the Federal Government or the basis on which appropriations for such work are to be secured. No doubt, a plan can be evolved which will not favor particular sections. Success will be greatly enhanced by general applicability.

Viewed in the light of the situation which would arise in the event that the public lands and minerals are ceded to the States, with elimination thereby of the original source of the Reclamation Fund, the Committee proposal merits careful consideration, especially by the Rocky Mountain States. Agriculture in the Lower Mississippi Valley is receiving a heavy subsidy in flood protection and cheapened water transportation. The Panama Canal gave Pacific Coast agriculture a decided advantage over the mountain region in reduced freight rates, while greater advantage is being sought under the guise of local aid to navigation.

Finding of Feasibility.—Careful investigation of new projects, as recommended by the Committee, will not mark a new departure for the Bureau of Reclamation. In earlier years, more stress was laid on engineering than land classification and economics, and rightly so, as works of a new magnitude and difficulty of construction were necessary. Agriculture was not then, nor

* *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1193.

for many years afterward, in a depressed state. With few exceptions the lands served with water could, at moderate cost, be placed on a profitable production basis.

The most unsatisfactory feature then was the inability to compel participation in costs by privately owned lands, the holders of which needed to pay only when using water. Irrigation districts with their powers of general assessment were introduced to meet this situation.

The agricultural situation of recent years and the less desirable quality of much of the land remaining to be irrigated have necessitated greater attention to matters of soil, land classification, and economics, so that these factors are now given consideration on a par with engineering. Considering the magnitude of the works undertaken and the human difficulties encountered, the results on Government projects compare very favorably with those obtained under private activity.

If projects had been accepted purely on their relative merits, the results to date would no doubt have been more favorable and the prospect of successful future development much brighter. The Bureau of Reclamation has not endorsed some projects undertaken in the past, and Congress has seen fit to modify the adopted 10-year program. Agitation for construction of projects without regard to their prospect of success has never been more pressing.

The proposal to have States assume responsibility in project selection has been tried by the Bureau through representation of the States on boards that reported on projects, often with disappointing results. Lack of training and, in some cases, exertion of political pressure and business influences precluded intelligent and unbiased consideration.

Construction Preference.—The suggestion by the Committee of favoring projects that will provide for supplemental water supplies for existing developments is, in fact, already the policy of the Bureau and with this policy Congress appears to be in harmony. Projects of this kind, when planned with due regard to proper stream utilization, will often result in the development of new lands.

The Gooding or Minidoka Gravity Extension Project is of this type. It was started as a Carey Act project more than twenty years ago, when a reservoir was constructed on Big Wood River and canal systems were built to irrigate the greater part of an area of 200 000 acres. Lack of water led to gradual shrinkage of the district area to 76 000 acres, of which about one-half is cultivated in scattered tracts, with a water supply inadequate for profitable farming. The only practical solution is to bring a supplemental supply from Snake River by means of a long, expensive canal to deliver stored water from American Falls Reservoir and, incidentally, provide the means of irrigating an area of new land as large as the old. While it is generally desirable to delay settlement of new lands (in this case largely public in character) the result is to load present settlers with the operation and maintenance cost of works intended for a much larger area, construction to less than full capacity being impracticable.

Suspension of Construction.—The agricultural depression of recent years has so seriously undermined the financial stability of innumerable privately

constructed developments, especially those with inadequate water supply or expensive operation, that wholesale abandonment can only be averted by reconstruction which they can not finance themselves and in the course of which some new lands will almost invariably be supplied with water. A general policy of discouraging, temporarily, construction that tends to increase crop production, is advocated by some as desirable. If adopted, the policy should not be limited to the arid States, but should be extended to all activities that tend to increase production, such as flood protection for overflowed lands in the Mississippi and Sacramento Valleys, drainage of swamp lands in all non-arid States, development of cut-over lands, and the rehabilitation of exhausted or eroded lands of the Eastern States. Such a policy requires careful consideration of the speed with which serious shortages in food supply can develop, while the preparation of new areas for production consumes much more time, especially in the case of irrigated lands; neither should a purely altruistic attitude of "the greatest good to the greatest number" be adopted in preparing the policy; the relative needs of various localities for development to maintain and improve their existence should be given careful consideration.

The continuation of development with the Reclamation Fund can hardly be considered a tax on other communities since two-thirds of the funds now used are derived from repayments by constructed projects, and this proportion is on the increase.

Construction Payments.—The Reclamation Bureau policy on the whole has never been other than that proposed by the Committee, although individuals within the Bureau, and possibly department heads, have wavered at times while certain members of Congress have publicly fostered resistance to such a policy and by every means at their command have aided and abetted evasion and reduction in payment of monies due the Government.

The crisis of 1921 and the subsequent years of agricultural depression were even more severe on the irrigated than on humid lands, as they were still in progress of development with credit resources strained to the limit. The most vulnerable creditor was the Federal Government. Relief in an acute situation, no doubt often well deserved, was found so easy to obtain, that a drive was then successfully made for more general relief. Although the relief granted may not have been justified in its entirety in all cases, the result has been such as to improve the morale of the settlers and to increase the revenue from these projects.

Much will be accomplished if the efforts by the Committee will aid in a new set-up that will remove the ever-present danger of a recurrence of these incidents.

Aided Settlement.—The Committee evidently has recognized that all is not well with the settlement of projects after water becomes available. It suggests stimulation by prevention of land speculation and recommends that financial aid should be a concern of the State and locality rather than the United States. The reference in the Committee report* to aid extended by

* *Proceedings*, Am. Soc. C. E., September, 1928, Papers and Discussions, p. 2100.

the Federal Land Bank is not pertinent, because the bank makes no loans on undeveloped farms.

In his discussion Dr. Mead describes* the gap between completion of construction and the use of water. The turning point, however, is not reached until water has been used a sufficient length of time and to a degree corresponding with farm development to that point where the operating income exceeds the annual costs and leaves a surplus for retirement of debt.

In earlier years, this gap was bridged by moderate amounts of hardship extending over a few years. A far different situation now exists, due, in part, to: (*a*) Lower quality of new lands left for reclamation; (*b*) increase of farm equipment required; (*c*) changes in living standards; and (*d*) decline in farming profits compared with other industries.

The first two items are promptly reflected in an enormous increase in the cash capital, or its equivalent, required at the outset. The latter two have resulted in an extremely low net income even on fully developed lands and have forced out marginal lands. The margin of profit is wholly lost under conditions of incomplete development and if such conditions extend over a series of years, failure is inevitable. Lack of adequate capital is the most common cause for failure.

To meet the situation, the Interior Department has adopted and is practising certain measures of general application that require no investment, such as: (1) Prevention of entries on public lands by unqualified settlers; (2) curbing of land speculation; (3) compulsory district organization to stimulate development of private holdings; (4) dissemination of information to attract qualified settlers; and (5) assistance in the sale of private lands to "dirt" farmers.

These measures have contributed notably to improvement in settlement, but they do not wholly meet the need of adequate capital. The present requisite of $2 000 liquid capital, or its equivalent in stock and equipment, for the settler on public lands is generally too small to permit a proper start. Analyses indicate the need for fully twice that amount. Further increase in the required capital would rapidly reduce the ranks of eligibles and would introduce resentment by untold numbers who still cling to time-worn ideas of carving a homestead out of public lands with little more than their bare hands for assets. On developments that provide supplemental water, lands are largely privately owned, but there is at present no means to compel selection of buyers when large holdings are broken up, and, apparently, there is no firm legal foundation that will permit imposition of such requirement. It must be said, however, that local interests have had marked success in endeavoring to eliminate unworthy purchasers. While these measures have been effective, they are far from adequate. Unsuccessful efforts have been made to obtain settlement funds from Congress for a trial demonstration. Support for this proposal is not strong in or out of Congress, due, no doubt, to a general conviction that the Federal Government has done enough in constructing the irrigation works and because of a misgiving as to the possibility of maintaing a proper business relation where one party has so little at stake.

* *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1208.

More than one effort has been made to obtain active and financial co-operation by States in settlement problems. In some cases no interest can be kindled, and nowhere has financial aid been forthcoming. In many quarters there is a decided tendency to consider the projects strictly a Government affair with all responsibility on the part of the State terminating when its representatives have brought about a start on construction.

The problem of settlement has not been solved; it warrants further serious study, because it will inevitably be a major factor in determining future reclamation policies.

Summing up the situation, the increasing difficulty of operations under the present plan and the growing criticism of the interest-waiver form of subsidy make it advisable to consider a change. The Committee's proposal of gratuitous construction of a part of the works has much merit and should be given further consideration. Settlement after completion of the engineering works is far from satisfactory and merits much more attention by those dealing with proposed changes in the Federal reclamation policy. State participation in construction and even more in settlement has been suggested, but all efforts to enlist the States in such work have been unsuccessful. If curtailment of development to reduce farm surplus is desirable and is adopted, it should be general and should be directed not only at irrigation activities, but impartially at irrigation, flood control, drainage, and all other activities throughout the country which either directly or indirectly result in increased production of those crops the overproduction of which is held to be responsible for depressed prices.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

PUMPED-STORAGE HYDRO-ELECTRIC PLANTS

Discussion*

By William W. K. Freeman, Assoc. M. Am. Soc. C. E.†

William W. K. Freeman,‡ Assoc. M. Am. Soc. C. E. (by letter).§—In his discussion,‖ Mr. John R. Freeman cites the proposal made by the late Julius M. Howells, M. Am. Soc. C. E., about 1905 to deliver power to San Francisco at a weekly load factor of 100% so as to cut down the investment in 200-mile transmission lines, fitting this power to the load curve through the action of a pumped-storage plant. This is one of five special applications of pumping which have been proposed to take up the slack between supply and demand. A second application is to the case of a navigable river where no pondage is permissible, in which case off-peak power may be converted into on-peak power in a similar manner. A third example is the proposed use of surplus blast-furnace or coke-oven gas to operate gas pumps of large capacity to store water in the off-peak hours.

In contrast to these cases in which power is available at a constant rate, two other special applications have been proposed to take power when it is available at variable times and deliver it to the load as required. One of these is being planned by Dexter P. Cooper, M. Am. Soc. C. E., at Haycock Harbor, Me., in connection with the Passamaquoddy Bay tidal project. The pumped-storage part of the project involves a reservoir with a net capacity of 13 000 000 000 cu. ft. for 40-ft. draw-down, with a maximum static head of 140 ft. The plant would operate on a monthly cycle, with 195 000 h.p. of pumping and 240 000 h.p. of turbine capacity. The advance beyond present record installations is shown by the proposal, among others, of one manufacturer for combination units, each unit with a turbine of 48 000 h.p. capacity and a pump of 39 000 h.p., delivering 2 700 sec-ft.

* Discussion on the paper by William W. K. Freeman, Assoc. M. Am. Soc. C. E., continued from August, 1929, *Proceedings*.

† Author's closure.

‡ Asst. Engr., New England Power Constr. Co., Boston, Mass.

§ Received by the Secretary, February 19, 1930.

‖ *Proceedings*, Am. Soc. C. E., February, 1929, Papers and Discussions, p. 529.

The other application is in connection with the development of wind-power plants. While no pumping feature is under consideration in the plans for the first wind-power plant, which will probably be completed in about a year, the engineers developing the plant have recognized the possibilities of a combination of wind power and pumped storage to convert off-peak into on-peak power.

The discussion by Mr. Foster is valuable in pointing out the use of the mass diagram (Fig. 7)* in the solution of pumping problems, and because he looks upon the addition of pumping equipment as one of a number of possible methods of developing power in a certain locality, and gives a clear picture of its economics.

Mr. Reed has added descriptions of four plants† to the list of those built. The writer was already indebted to him for his description of Lüner Lake and three others.‡ Mr. Reed's emphasis on the stand-by feature of pumping plants in Germany is in line with the point of view in this country for some of the proposed developments. The rapid progress of interconnection in the United States, however, probably removes this development of pumping plants to a later date when the older steam plants which are now held in reserve, will have been scrapped.

Mr. Heslop also emphasizes the reserve value of pumping plants.§ As a matter of fact, the first use of the Rocky River Plant was as a reserve plant, because as soon as the operating engineers of The Connecticut Light and Power Company were informed that the plant was ready for limited service, even before the reservoir was filled, the operation of reserve boilers at the Devon Steam Plant was discontinued.

Mr. Markwart's discussion‖ is exceptionally valuable because it includes curves of justifiable investment cost for plants of the pondage type. These curves can be used as guides to determine which projects are worth further investigation and which are obviously uneconomical. The writer's experience, while limited, bears out the statement¶ that, "because of the remarkable over-all efficiency of modern steam plants, pumped-storage plants will be of no value as an aid to composite economy in power production unless the capital cost per kilowatt of installed capacity is relatively low".

The curves, Figs. 10,** 11,†† 12,†† and 13‡‡ show the limiting cost to be about \$120 to \$150 per kw. for systems with base-load plants burning coal that costs, respectively, \$4 to \$8 per ton. In October, 1929, Professor Dr.-Ing. D. Thoma, Director of the Hydraulic Institute, Technische Hochschule, Munich, gave the limiting cost in Germany as \$80 per h.p., equivalent to \$113 per kw., which checks roughly Mr. Markwart's figures.

* *Proceedings*, Am. Soc. C. E., February, 1929, Papers and Discussions, p. 532.
† *Loc. cit.*, March, 1929, Papers and Discussions, p. 820.
‡ *Loc. cit.*, November, 1928, Papers and Discussions, p. 2475, Reference (9).
§ *Loc. cit.*, May, 1929, Papers and Discussions, p. 1277.
‖ *Loc. cit.*, August, 1929, Papers and Discussions, p. 1617.
¶ *Loc. cit.*, p. 1619.
** *Loc. cit.*, p. 1630.
†† *Loc. cit.*, p. 1631.
‡‡ *Loc. cit.*, p. 1632.

Since projects costing only $120 to $150 per kw. are rare, Mr. Markwart's calculations bear out the writer's opinion that pumping plants of the pondage type do not appear to be as attractive at the present time as those of the storage or part-head types, unless perhaps they are built in conjunction with existing developments. It is the storage feature of Rocky River which makes it economical in spite of the comparatively low total head through which the stored water is used; this head averages 285 ft., excluding old hydro-mechanical developments. The Rocky River Plant has several other advantages, moreover, which help to make it economical; the water supply for its pumps is drawn from a drainage area of 1 037 sq. miles, and the power house is as close to the load center as is the base-load steam plant at Devon; that is, about 30 miles. Certain features of topography are also in its favor; the reservoir basin was partly swamp and abandoned farm land, and the fall of the Rocky River is largely concentrated into the mile above its mouth, so that the total volume of earth in the dam and dikes is only 750 000 cu. yd.

The exceptional value of plants of the pondage type for emergency service may in time make them more attractive, but at present capacity is "a drug on the market" in many localities because of the rapid progress of interconnection which has made many small steam plants available for stand-by service.

The writer was able to check points on Mr. Markwart's curves of justifiable investment cost by following the usual method of comparison with an alternate steam plant, as outlined by the writers* and amplified by Mr. Markwart under his six premises.† Mr. Markwart states‡ that his study is methodical only, and warns against using his curves for individual situations. This warning the writer would emphasize by adding that even for a certain power system at least one change in the method of analysis appears to be necessary.

This relates to the sizes of hydro-electric and steam plants assumed to be alternative. Mr. Markwart states§ that,

"In the case of a small pumped-storage plant a relatively high capital expenditure per kilowatt of capacity would be justified because its alternate steam plant would have a high capital cost and a relatively low fuel economy".

Throughout his discussion of capacities, steam pressures, and fuel economy, Mr. Markwart assumes that these must be selected for a steam plant of the same size as the pumping plant under discussion. This is true only in the broadest generalization, because the selection of capacity for a pumping plant depends essentially on the topography, while selection of capacity, etc., for a steam plant depends on the characteristics of the system and the rate of growth of its load. Thus, for a system in which 50 000-kw. steam units are being installed, the capacity, steam pressure, fuel economy, operation and maintenance charges, and load factor must be that of the 50 000-kw. alternative steam unit whether the pumping plant is of 5 000 or of 200 000 kw. The difference in interest charges on unused capacity until the larger of the alternative plants is fully loaded, is applied to the construction cost in arriving at the justifiable investment.

* *Proceedings*, Am. Soc. C. E., November, 1928, Papers and Discussions, p. 2460.
† *Loc. cit.*, August, 1929, Papers and Discussions, p. 1625.
‡ *Loc. cit.*, p. 1636.
§ *Loc. cit.*, p. 1626.

Another change in the method of analysis which would be necessary for a particular case would be the inclusion of an increment operation and maintenance cost in addition to the coal cost given by Mr. Markwart in Table 4,* in calculating the energy charges of an installation. This cost is made up of coal storage and boiler and turbine maintenance; it equals about 0.3 mill per kw-hr. in a plant of 250 000 kw. capacity, and probably more for smaller plants. A closer analysis would also require the selection of economies, as given in Table 4, not only for size of plant, but also for load factor, instead of using one figure for everything from 10 to 100% load factor.

In conclusion, there are a few matters to be mentioned to bring the subject more nearly up to date. The Rocky River pumps, which were guaranteed to have 87% efficiency, have been tested since installation and a maximum efficiency of 91.9% was obtained with both pumps running under a dynamic head of 230 ft. This is far more than the previous maximum of 85% obtained at Schwarzenbach, Germany.

The term, "pumped-storage plant," has found a certain measure of acceptance. The ambiguous term, "hydraulic accumulator," has been observed only in a digest of a German article translated too literally. Other terms which have been proposed as a substitute for the writer's "pumped storage" (which is a free translation of "Pumpen-Speicherkraftwerk") are: (1) "Water storage battery"; (2) "hydro-regenerating plant"; and (3) "pumping plant". The first of these might be considered ambiguous because of its possible association with a wet-cell electric storage battery, and the second lacks the brevity of the third. The writer inclines to "pumping" instead of "pumped storage" because of its brevity, where it is obvious what kind of pumping plant is meant.

Three important additions to the bibliography should be noted. Joel D. Justin, M. Am. Soc. C. E., has presented a paper† entitled, "The Rocky River Plant of the Connecticut Light and Power Company"; and Paul L. Heslop, M. Am. Soc. C. E., a paper‡ entitled, "A Hydro-Electric Plant That Pumps Its Own Water Supply—The Rocky River Hydro-Electric Plant." Mr. Heslop has also written an article on the Rocky River pumps.§

The writer would like to thank those who have assisted in the preparation of the paper or contributed to the discussion. He welcomes this opportunity especially to acknowledge his indebtedness to Mr. Justin for much more than the "invaluable criticism" which was attributed to him in spite of his suggestion of anonymity. Practically every idea on the economics and operation of pumping plants which appeared in the paper was obtained directly or indirectly from Mr. Justin, and the writer is glad to pay a tribute to him as his generous and kindly mentor and former chief.

* *Proceedings*, Am. Soc. C. E., August, 1929, Papers and Discussions, p. 1629.

† *Loc. cit.*, March, 1929, Papers and Discussions, p. 690.

‡ *Proceedings*, Connecticut Soc. of Civ. Engrs., 1928, p. 6.

§ "Two 8 100 H. P. Pumps Supply Water for Power Generation," *Power*, December 24, 1929, p. 1006.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

PROBLEMS CONCERNING ELASTIC STABILITY IN STRUCTURES

Discussion*

By S. Timoshenko, Esq.†

S. Timoshenko,‡ Esq. (by letter).§—The purpose of the paper was to give a brief discussion of several problems of elastic stability, different from simple column problems and of interest to structural engineers. In all cases it was assumed that the bars were slender and the plates thin, so that buckling occurred within the elastic limit of the material. There are many cases of more complicated conditions in which buckling occurs beyond the elastic limit, and the writer completely agrees with Professor Westergaard's statement‖ that these problems "represent a field for future research, both experimental and analytical."

In the case of trusses, the compressed members are usually in a condition of eccentric compression, due to rigid joints. The magnitude of the eccentricities can be calculated by the methods used in the analysis of "secondary stresses" in trusses. Having these eccentricities it seems logical to design these members as bars submitted to the simultaneous action of compression and bending. As an example, take the simplest case, when the eccentricities on both ends are equal and in the same direction. The maximum bending moment at the middle is then given by the well-known secant formula:

$$\mu_{\max.} = P_e \sec \frac{l}{2}\sqrt{\frac{P}{EI}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (41)$$

in which, P is the eccentrically applied compressive force, and e, the magnitude of the eccentricities.

* Discussion of the paper by S. Timoshenko, Esq., continued from November, 1929, *Proceedings*.

† Author's closure.

‡ Ann Arbor, Mich.

§ Received by the Secretary, February 15, 1930.

‖ *Proceedings*, Am. Soc. C. E., September, 1929, Papers and Discussions, p. 1891.

The maximum compressive stress is:

$$s_{\max.} = \frac{P}{A} + \frac{\mu_{\max.}}{S} = \frac{P}{A}\left(1 + \frac{e}{r}\sec\frac{l}{2}\sqrt{\frac{P}{EI}}\right) \quad \ldots\ldots (42)$$

in which, $r = \frac{S}{A}$.

In using Equation (42) to determine cross-sectional dimensions, it is important to note that the maximum stress is not proportional to the load, P, but increases more rapidly than the load. Hence, it is not logical in Equation (42) simply to make $s_{\max.}$ equal to the working stress, s_w, when designing the bar. This is explained by Fig. 18 which shows $s_{\max.}$ as a function of P, as given by Equation (42). This curve has as its asymptote the vertical line, BC, such that,

$$OB = P_{cr} = \frac{EI\pi^2}{l^2} \quad \ldots\ldots (43)$$

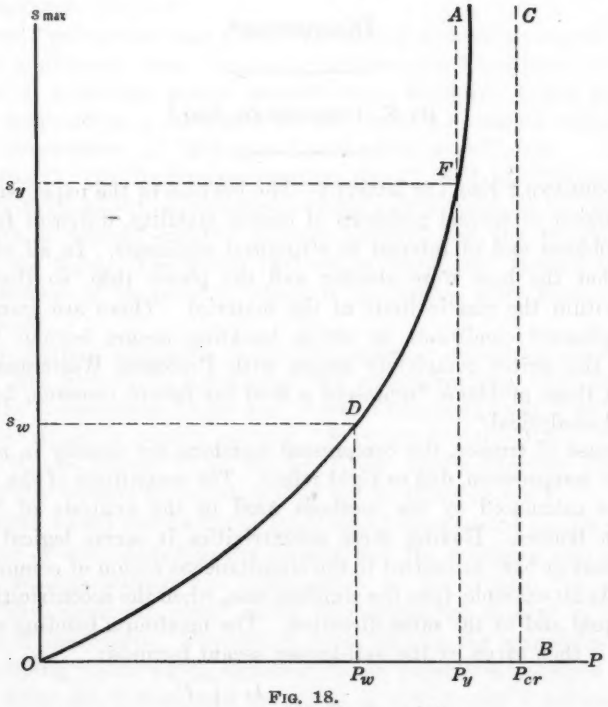


FIG. 18.

Select a working stress that is a certain amount less than the yield-point stress, s_y, for example, let $s_w = \frac{s_y}{2}$. In Fig. 18, this corresponds with the point D, on the curve and it follows that the load is P_w. It will be seen that, due to the non-linear relation between $s_{\max.}$ and P, the ratio, $\frac{P_w}{P_y}$, is different from the ratio, $\frac{s_w}{s_y}$.

While s_w is one-half s_y, P_w is more than one-half P_y, and the factor of safety with respect to the beginning of yielding is smaller than 2. In order to arrive at the prescribed factor of safety, let s_w and s_y represent, respectively, the working stress for short compressed bars and the yield point of the material, and let n be the factor of safety. This factor, then, must be such that the maximum stress, given by Equation (42) will equal the yield point when the load, P, is increased n times. It is assumed that Equation (42) is accurate enough to describe conditions also when $s_{\max.}$ approaches s_y. For this limiting condition, Equation (42) becomes:

$$s_y = \frac{n\,P}{A}\left(1 + \frac{e}{r}\sec\frac{l}{2}\sqrt{\frac{n\,P}{E\,I}}\right) \qquad (44)$$

from which,

$$\frac{1}{n}\,s_y = s_w = \frac{P}{A}\left(1 + \frac{e}{r}\sec\frac{l}{2}\sqrt{\frac{n\,P}{E\,I}}\right) \qquad (45)$$

Comparison of Equation (45) with Equation (42) shows that the replacement of P by nP, under the radical, takes proper account of the fact that $s_{\max.}$ increases faster than P.

If P, e, n, and s_y are known, the cross-sectional dimensions may be determined by cut-and-try methods so as to satisfy Equation (45). Manipulation with this equation can be greatly simplified by preparing tables for $\frac{P}{A}$ for various values of n and $\frac{e}{r}$ and for various slenderness ratios of struts. Such tables were calculated for several cases by K. S. Zavriev* and an analogous method was proposed by A. Ostenfeld.†

An analogous analysis can be developed for any other condition of eccentricity. With the tables mentioned, compressed members with any given factor of safety can easily be designed. This method of treating compressed members also gives a logical basis for dimensioning lattice-bars of built-up columns. These bars must be strong enough to sustain the shearing force, the value of which is easily derived when the eccentricities in the application of the compressive force are known.

Both methods of reasoning applied by Professor Hovgaard in deriving Equations (27)‡ and (28)‡ are legitimate and both of them reach the same final result. This subject was discussed elsewhere in detail by the writer§ and he did not consider it necessary to repeat it.

* *Bulletins*, Soc. of Engrs. of Technology, 1913, St. Petersburg.
† *Ingeniorvidenskabeligeskrifter*, 1929, Copenhagen.
‡ *Proceedings*, Am. Soc. C. E., September, 1929, Papers and Discussions, p. 1894.
§ *Zeitschrift für Math. u. Phys.*, Vol. 58, 1910, p. 337.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

THE SURVEYING AND MAPPING ACTIVITIES OF THE FEDERAL GOVERNMENT

Discussion*

By T. W. Norcross, Assoc. M. Am. Soc. C. E.†

T. W. Norcross,‡ Assoc. M. Am. Soc. C. E. (by letter).§—The co-ordination between the surveying work of the General Land Office and of the U. S. Coast and Geodetic Survey is cited by Mr. Kidder.‖ This instance is only one of many brought about by or through the Federal Board of Surveys and Maps.

Mr. Bowie mentions¶ the co-ordination of the Federal surveying activities and the accuracy of the governmental work. Concerning the work of cities, States, counties, and private corporations, he asks whether the conditions are as satisfactory as in the Federal work. From present information, the writer agrees that the answer must be decidedly negative. Probably no one questions the advisability of thorough co-ordination of non-Federal surveying activities. How can this be accomplished? A more general acceptance and use of the standards established by the Federal Board of Surveys and Maps, where such standards are applicable, would help in the matter of accuracy. How to co-ordinate the efforts is a question that could well be studied by the Surveying and Mapping Division.

Mr. Tufts** and Mr. Bowie†† discuss the map situation in the United States. The value of maps has been clearly stated by L. R. Lohr,†† M. Am. Soc. C. E. Engineers and certain map users agree that there is a real need for more accurate maps and urge greater speed in mapping the United States.

* Discussion on the paper by T. W. Norcross, Assoc. M. Am. Soc. C. E., continued from September, 1929, *Proceedings*.

† Author's closure.

‡ Chf. Engr., U. S. Forest Service, and Chairman, Federal Board of Surveys and Maps, Washington, D. C.

§ Received by the Secretary, February 6, 1930.

‖ *Proceedings*, Am. Soc. C. E., September, 1929, Papers and Discussions, p. 1902.

¶ *Loc. cit.*, p. 1899.

** *Loc. cit.*, p. 1901.

†† *Loc. cit.*, April, 1929, Papers and Discussions, p. 887.

The sentiment in favor of more and better maps should be unanimous; but apparently it is not. Much has been said of the failure of the Federal Government to provide sufficient appropriations. Has sufficient consideration been given to the mapping that clearly is not a Federal function or obligation? In one State, for instance, the topographic mapping is anly about one-third completed; yet the State expenditures for a recent year were less than $900. This does not indicate that, in the opinion of engineers in that State, the value of topographic maps is great, that the need is urgent, and that engineers and map users are not satisfied to use maps which they know are inaccurate or do not have a degree of accuracy consistent with the use. Evidently, a need exists for the education of large numbers of map users in the value and proper use of maps.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

GOOD MAPS ARE A SOUND INVESTMENT

Discussion*

By L. R. Lohr, M. Am. Soc. C. E.†

L. R. Lohr,‡ M. Am. Soc. C. E. (by letter).§—Since this paper has called forth discussions from only six engineers and geographers‖ who apparently agree with the views of the writer and who have sought only to add emphasis to the expression of those views, there appears to be little doubt regarding the value of good maps, in the minds of those who know them and their applications to the practical problems of everyday life. Further discussion thus becomes repetition, serving only to emphasize views already expressed.

Known facts are the only working basis from which unknown factors or results may be logically determined. A complete knowledge of the facts regarding any undertaking is essential to its successful accomplishment. Maps are tabulations in graphic form of the facts of terrain. Good maps present a picture of all such facts in their proper relation one to another, within the limits of the scale and the type of the map. Accurate maps, because of the complete and correlated facts presented, are essential to practically all engineering and industrial work. Such maps must not only be made, but their existence must be known, they must be available when needed, and engineers and the public generally must be able to read the facts which they present. The accomplishment of this will require a sustained, organized effort which might well be sponsored and directed by the Society.

The necessity of good topographic maps for engineering, industrial, and administrative purposes should be stressed to the legislative and administrative branches of the State and Federal Governments. Map reading should be more generally and more thoroughly taught in the schools and colleges.

* Discussion of the paper by L. R. Lohr, M. Am. Soc. C. E., continued from September, 1929, *Proceedings*.

† Author's closure.

‡ Maj., Corps of Engrs., U. S. A.; Mgr., Chicago World's Fair Centennial Celebration, 1933, Chicago, Ill

§ Received by the Secretary, February 27, 1930.

‖ *Proceedings*, Am. Soc. C. E., September, 1929, Papers and Discussions, p. 1905.

Existing maps should be classified and indexed in a manner that will show the area covered, the character and scale of the map, the place where it may be obtained or consulted, the purchase price if for sale, or any other pertinent facts that may serve the public generally. This should be done for each community, and a copy of the classification and index should be furnished to some central agency, either established or to be established, which will serve as a clearing house for map information. Such an agency now exists in the Map Information Office, Board of Surveys and Maps, Interior Building, Washington, D. C. The Map Information Office receives considerable co-operation from non-Federal agencies, but there is not as much as there should be to enable it to render satisfactory service in all cases. With complete information centralized in an office of this character, much duplication of effort and expense could be eliminated, and preliminary engineering and industrial studies could be expedited.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

WATER SUPPLY FROM RAINFALL ON VALLEY FLOORS

Discussion*

By Charles H. Lee, M. Am. Soc. C. E.

Charles H. Lee,† M. Am. Soc. C. E. (by letter).‡—The author's treatment of deep penetration of rainfall is developed primarily for determination of the increment to water supply in the alluvial valleys of Southern California. The subject is one of considerable interest, however, in all semi-arid districts where growth of population has created a pressing demand for water. It may also be important in desert valleys, because the latter are usually surrounded by mountain ranges receiving more generous precipitation, a portion of which falls upon the marginal slopes of the valley. Although deep penetration from rainfall has been recognized as a source of sub-surface water, no definite methods have been generally adopted for quantitative determination on a large scale. The demand for such methods now exists, and this paper represents an initial step in filling this need.

At the outset, in the consideration of the paper, it should be recognized that the character of alluvial deposits as generally found in valleys does not conform with the author's description and illustration on Fig. 1.§ It is true that in many of the alluvial valleys of Southern California there are deposits which "are essentially heterogeneous in formation". These are the product of deposition from steep graded streams and result from reduced velocity due to flattening of slope. For these deposits the author's concept may be correct.

There are many other alluvial filled valleys and parts of valleys, however, throughout California and, in fact, generally throughout the world, in which homogeneity in formation is the ruling characteristic. Such valleys at one time or another in their history have experienced temporary flooding, or have

* Discussion of the paper by A. L. Sonderegger, M. Am. Soc. C. E., continued from January, 1930, *Proceedings*.

† Cons. Hydr. Engr., San Francisco, Calif.

‡ Received by the Secretary, January 25, 1930.

§ *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, pp. 1140–1141.

been occupied by large lakes or arms of the sea which have had wide fluctuation in level. In these valleys, stream deposition has resulted either from the sudden checking of velocity at the edge of standing water, or from flattening of grade at the margin of an unwatered lake bed. These conditions have resulted in segregation of alluvial material into extensive sheets or beds of distinctive character. The beds consist of fine sands, silts, and clays, deposited from still water upon the lake bottom, and coarse sands and gravel derived from rehandled and segregated shore deposits which have been spread over the lake bottom by flowing water at periods of low lake stage. The occurrence of homogeneous valley formations is so general that the discussion of the paper would be incomplete if mention were not made of them.

Another concept presented by the author, which is stated without qualification, is that because of the heterogeneity of alluvial deposits the movement of absorbed water from the ground surface to the water-table is by way of "more or less defined ducts", and that it is "likely to collect into trickling streams".*

Although the condition described by the author may be characteristic of the steep boulder and gravel slopes surrounding certain Southern California valleys, yet it is not a general condition. Most valleys have a soil cover which is of homogeneous texture and is underlaid by a subsoil of similar texture. In such formations the downward movement of water is generally through capillary interstices, and is more or less uniformly distributed. It frequently occurs that roots will open up a soil and, when rotted, will leave tubes which assist in the more rapid downward penetration of water; but this is merely an aid to penetration, and in no case does it extend to great depth.

The mechanics of rainfall penetration are so important to an understanding of the problems considered by the author, that the writer believes a short description would not be amiss. If rain falls upon soil, with moisture content at or exceeding field capacity, there will be an immediate gravity movement of water by way of fat capillary films to the water-table, and an observable rise of the latter. If, however, the soil has been depleted of moisture below the field capacity, penetration to the water-table does not occur unless and until complete connection has been established by capillary films extending from the surface to the water-table.

As the advance of moisture by capillarity alone is extremely slow, it frequently happens, in regions where the annual depth of rainfall is small as compared to the annual depth of water evaporation, and in areas where the depth to the water-table exceeds the capillary range, that little or no water will reach the water-table during a whole season. The reason for this is that the losses by run-off interception, soil evaporation, and transpiration are so great that most or all water falling as rain is consumed either before or after penetrating the soil.

This point is well illustrated by observations made by the writer at twenty-four wells in the river valleys of San Diego County, California, before and

* *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, pp. 1140–1141.

after the first heavy storm of the season, 1914-15, beginning January 27, 1915.* The formation in these valleys is of uniform textured beds of fine to coarse sand, the top stratum containing more or less silt. The soil and sand above the capillary fringe was very deficient in capillary moisture at the end of nine months' rainless period. It was found that at all wells in which the depth to the water-table, just prior to the storm, was less than the distance to which capillary water would rise (approximately 7 ft.), the effect of the storm on the water-table was pronounced and reached its maximum within from 2 to 7 days. For wells in which the depth exceeded 7 ft., no pronounced fluctuation was observed although after a short time had elapsed a gradual rise was noted culminating in from 20 to 60 days. The range of depth to water-table in the latter wells was from 7.4 to 19.4 ft. A very sharp difference was evident, and there was no difficulty in grouping the wells.

Comprehensive observations have been made by Walter E. Spear, M. Am. Soc. C. E., of the deep penetration from rainfall in rehandled glacial deposits of Long Island, New York, where the soil and subsoil are maintained more or less at field capacity throughout the year by frequent storms.† More than fifty wells were studied, with widely differing depths to water-table, which ranged from 1 ft. to 95 ft. The material through which moisture percolated consisted of 12 to 18 in. of brown loam, 18 to 24 in. of yellow subsoil, and glacial outwash made up of coarse to fine sand occasionally mixed with coarse and fine gravel, and very fine sand. For depths to water-table of 7 ft. accretions from rainfall were registered at from 1 day to 4 days, which corresponds closely with the writer's results in San Diego County. For a depth of 20 ft., the time interval was from 4 to 14 days. In comparison with the latter, the time required in San Diego County wells with deficient capillary moisture was from 20 to 60 days. The effectiveness of soil moisture content at field capacity in facilitating deep rainfall penetration is thus clearly apparent.

The importance of regular sequence and magnitude of storms in producing deep penetration in regions with distinctive wet and dry periods, which has been so well brought out by the author in his discussion of general conditions,‡ are further emphasized by the data contained in Mr. Spear's report. Depletion of soil moisture always occurs during long periods without rain, unless a permanent water-table lies within capillary distance of the surface. In an area where the water-table lies more than from 7 to 10 ft. below the surface, a certain volume of water is required to satisfy capillary demands before penetration to the water-table will occur. This volume, and the time required, increases with the depth to the water-table. In order that penetration to the water-table shall occur, the rainy season must be long enough, and the storms sufficiently frequent and large to bring the moisture-depleted soil and subsoil up to field capacity from the surface to the water-table prior to the end of the season. The extent to which moisture reaches the water-table through "well-

* *Water Supply Paper 446*, U. S. Geological Survey, 1919, by A. J. Ellis and Charles H. Lee, p. 140, Table 35.

† Report of Commission on Additional Water Supply for the City of New York, 1903, Appendix VII, by Walter E. Spear, Plate VI and Table 25, pp. 798 to 805.

‡ *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, pp. 1140–1144.

defined ducts" or "trickling streams" is under ordinary conditions extremely problematical, and the writer believes cannot be depended upon to speed up the process except in rare cases, such as that which the author seems to have encountered.

Determination of Rainfall Penetration from Soil Moisture Tests and from Springs in the Murrietta-Temecula Area.—The author assumes,* for the purpose of reaching quantitative conclusions, that deep rainfall penetration from a certain mesa area underlaid by compact stratified alluvium is the only source of supply for a group of springs and flowing artesian wells. The latter are located at the base of a steep slope terminating the mesa. To prove that deep penetration occurs, he presents soil moisture records at test plots which are enclosed by embankments to prevent surface run-off. These records indicate that of 14.82 in. depth of rainfall and water applied to the plots between February 4 and March 7, 1927, 3.31 in. were unaccounted for within the first 9 ft. depth of soil, if evaporation loss from surface soil is assumed at 2.00 in. (see Table 5).† The author concludes that this water penetrated deeply and reached a water-table approximately 200 ft. below the surface of the mesa. As proof, he presents water-level observations in four wells, three of which show a slight rise in the late spring of 1927 (see Table 7).‡ He presents no proof, however, that there is a general water-table under the mesa, or that the springs and artesian wells have any relation to a general water-table.

The writer believes that in a few special cases this method may be useful in obtaining quantitative data of deep percolation, but that it must be used with caution. In the example presented by the author,§ there are several points on which the facts submitted are incomplete or not in conformity with the conclusions. For instance, assuming that percolation to a depth greater than 9 ft., occurs from a few isolated depressions on the surface of the mesa, does it occur generally over the whole area, and is all water penetrating below 9 ft. beyond the reach of capillarity and plant roots? The well observations presented in Table 7 are not conclusive on this point. Well 129, near the test plots, does not show any rise. Well L8N, farther down the slope, shows 0.25 ft. rise in 27 days; and Wells 131 and 14, still farther down the slope, show rises of 1.30 ft. in less than 90 days and 1.10 ft. in 47 days, respectively. Thus, the greatest rise occurred at the lower edge of the slope of the assumed water-table, and a profile would show a flattening of the slope during the period when the author assumes that it is being replenished by deep percolation. Such flattening is, of course, an indication of depletion rather than of replenishment.

Considering another angle, is it an entirely reasonable assumption that water would move downward in less than 90 days through 200 ft. of moisture depleted material composed of compact stratified alluvium containing extensive clay beds?‖ In the test made by Mr. Spear on Long Island where con-

* *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1144.
† *Loc. cit.*, p. 1148.
‡ *Loc. cit.*, p. 1151.
§ *Loc. cit.*, p. 1144–1152.
‖ *Loc. cit.*, p. 1150.

ditions are most favorable for quick penetration, it required from 89 to 109 days for moisture to reach a depth of 80 ft. Is it not a more probable assumption that, if penetration occurred beyond the depth of capillary return, the water would move laterally as soon as it reached an impervious clay bed, and escape into some ravine or follow an old buried stream channel to the margin of the alluvium? The observed rise of water level in wells might possibly have been a rise due to pressure increase and not water-table rise.

The author does not present the data upon which he determines the area from which the flow of the springs and wells is derived. Due to the varied structure of the alluvial formation, this would be subject to considerable uncertainty unless fairly complete geological and hydraulic facts were available. It is quite possible, for example, that the springs and wells represent the discharge from the open end of an old buried stream channel interbedded between clay strata instead of from a general water-table. If this is the case, the tributary area is impossible of determination from an examination of surface topography alone. The fact that there is not an extensive spring or seepage horizon along the base of the slope adjacent to the springs would support such an assumption.

The uncertainties, previously mentioned, indicate some of the difficulties involved in the practical application of this method, and they point to the fact that the author's quantitative result, while suggestive, is not the final answer.

Determination of Rainfall Penetration by Comparison with Mountain Run-Off.—The method of determining rainfall penetration by comparison with mountain run-off is based on two assumptions: First, that for mountain water-sheds underlaid by impervious bed-rock which extends to the point of stream measurement, surface run-off throughout the annual cycle will correspond to the sum of surface run-off and deep percolation in adjacent valley water-sheds; and, second, that rainfall run-off relations for adjacent mountain and valley water-sheds are so similar that necessary modifications due to differences in soil cover, etc., can readily be made.

The writer believes that this method has possibilities for general use, but that its successful application will depend upon quantitative evaluation of the various factors which govern the rainfall run-off relation, rather than dependence on geographic proximity of water-sheds.

The factors upon which the rainfall run-off relation depend, can be grouped as meteorological and physical. Among the important meteorological factors are: (1) Precipitation; (2) temperature; (3) evaporation from water and snow surfaces; and (4) length of the growing season.

The physical features of water-sheds, which are of importance, are: (1) Elevation; (2) topographic catchment area; (3) topography; (4) soil and subsoil; (5) geology; (6) vegetation; and (7) cyclic storage capacity.

The precipitation falling upon a water-shed is disposed of either by: (1) Storm or seepage run-off, the latter being represented by deep percolation in valley water-sheds; (2) evaporation from water intercepted by foliage and branches of vegetation; (3) evaporation from snow or from soil moistened by

direct precipitation, melting snow, or capillary water held in the soil; (4) transpiration from growing vegetation; or (5) sub-surface leakage. Run-off curves, when prepared quantitatively in terms of depth of precipitation and run-off, as in Fig. 8,* segregate these various items into two groups, as follows:

For Mountain Water-Sheds:

1.—Storm and seepage run-off.
2.—Evaporation, transpiration, sub-surface leakage (usually negligible).

For Valley Water-Sheds:

1.—Storm run-off.
2.—Evaporation, transpiration, seepage run-off, or sub-surface leakage.

If the valley occupies a closed basin, and storm run-off is measured at the outlet, sub-surface leakage will be small or negligible, represented by the underflow of the stream. For open valleys, sub-surface leakage may be large, although it does represent a definite quantity of water that can be salvaged by pumping from wells and, therefore, may be considered as potential water supply.

The quantitative rainfall run-off curve, for any water-shed, presents graphically the integrated effect of the four meteorological and seven physical factors listed previously. From this curve can be measured, for any depth of annual precipitation, the corresponding depth of evapo-transpiration loss plus sub-surface leakage; minor corrections are sometimes necessary for cyclic storage. For mountain and closed valley water-sheds this quantity represents the evapo-transpiration loss alone. Its value has one of two absolute upper limits, either the annual depth of evaporation from a large free water surface (if that is less than the annual depth of precipitation), or the annual depth of precipitation, if the latter is less than the evaporation. Actually, the evapo-transpiration loss is less than either of these quantities, being limited by the evaporation opportunity.

In order to give perspective in analyzing the weight to be assigned various factors which influence the evapo-transpiration value for mountain water-sheds, the writer has found it useful to study water-sheds with widely differing meteorological and physical characteristics. As an illustration, the rainfall run-off curves of five mountain water-sheds are presented on Fig. 15. A statement of the corresponding water-shed characteristics is given in Table 18, and the trunk stream profiles are shown on Fig. 16.

The values for Item 15 of Table 18 were determined graphically from Fig. 15, by making necessary corrections to allow for the actual slope area. The temperatures given in Table 18 (see Items 8, 9, and 10) are based on U. S. Weather Bureau records obtained at stations representative of the water-shed area. Item 16 for the different water-sheds was determined in various ways as follows:

(*a*) For Coquille River an estimate was made from an incomplete, 4-year record obtained by the U. S. Weather Bureau at Corvallis, Ore.

* *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1156.

(*b*) The Rogue River evaporation was estimated from a 4-year floating-pan record at Klamath Falls, Ore.* (Elevation 4 100), and from an incomplete 4-year record at Fish Lake, Oregon† (Elevation 4 687).

(*c*) The average annual precipitation for Devil's Canyon Creek was estimated as approximately 55 in.

(*d*) The corresponding value for the San Luis Rey River is the average of a 3-year floating-pan record at Big Lake, near Warner's Hot Springs, Calif.‡ (Elevation 2 780).

(*e*) For the San Diego River, an average was taken from the 3-year floating-pan records at La Mesa Reservoir (Elevation 480) and Cuyamaca Reservoir§ (Elevation 4 620).

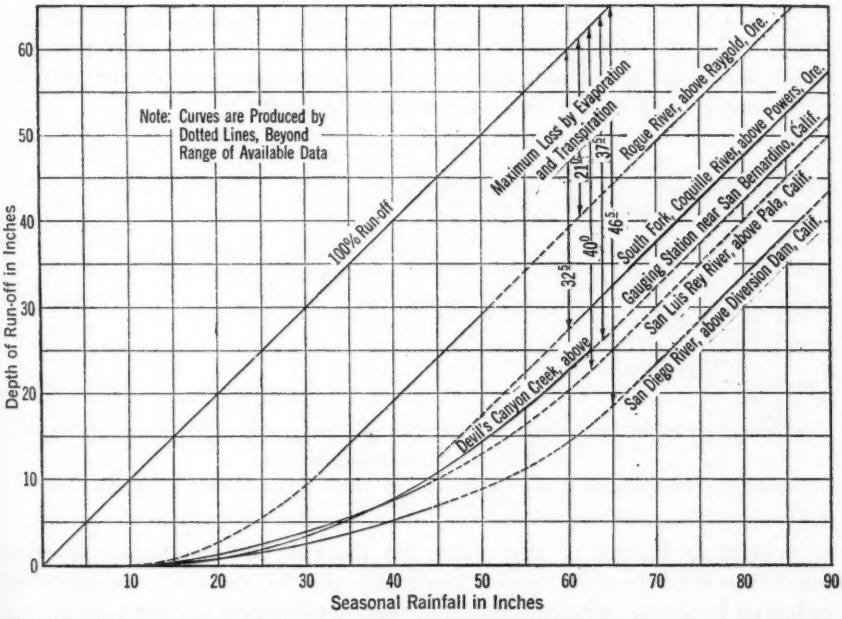


FIG. 15.—COMPARISON OF RAINFALL-RUN-OFF CURVES FOR STREAMS IN SOUTHERN OREGON AND IN SOUTHERN CALIFORNIA.

For the South Fork of Coquille River and Rogue River in Southwestern Oregon, precipitation equals or exceeds evaporation; while for San Luis Rey and San Diego Rivers, and also Devil's Canyon in Southern California, the reverse is true. Precipitation occurs largely as snow on the Rogue River, while snow falls only upon the higher parts of the other water-sheds. The bed-rock formations also differ. Sedimentary rock underlies the Coquille, while the Rogue is largely covered by deep lava flows resting upon crystalline rock. The Southern California water-sheds are of granitic formation. Wide

* *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 273.
† U. S. Weather Bureau, Climatological Data, Oregon Section, Years 1923 to 1928.
‡ *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), p. 1909.
§ *Water Supply Paper 446*, U. S. Geological Survey, p. 102.

differences in vegetational cover also exist. The Coquille supports a very dense forest growth, largely Port Orford cedar and Douglas fir, with alder and luxuriant shrubbery and undergrowth filling open spaces; the Rogue has open forest cover of pine and fir; and the Southern California streams have dense brush cover with alder, sycamore, and willows along stream bottoms.

Precipitation data for the preparation of all rainfall run-off curves have been obtained by planimeter measurement from detailed isohyetose maps of average seasonal precipitation corrected in each year by the index of seasonal wetness as derived from five or more base precipitation stations. Most of the latter have twenty-five years or more of record. Run-off data are available for periods of 10 to 25 years on the various streams. All precipitation, run-off, and drainage-area data are from the records of the U. S. Weather Bureau and U. S. Geological Survey. The curve for Devil's Canyon Creek as thus drawn (Fig. 15), differs from that of the author (Fig. 8). The principal reason is that the writer used a greater depth of average seasonal precipitation which seems to be supported by all available records.

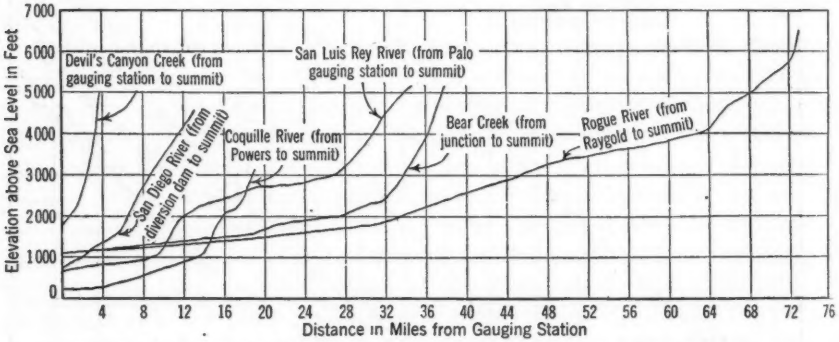


FIG. 16.—PROFILES OF DRAINAGE AREAS FOR STREAMS IN SOUTHERN OREGON AND SOUTHERN CALIFORNIA.

A striking feature of the curves for the two Oregon streams is their parallelism with the 100% run-off line within the full range of observed data, including both very wet and very dry years. (Compare the solid lines; the curves are produced by dotted lines beyond the range of available data.) This is due to the fact that even the minimum seasonal precipitation exceeds the maximum evapo-transpiration demand. For Devil's Canyon water-shed the upper end of the curve is just coming parallel to the 100% run-off line at the 60-in. rainfall, indicating that the precipitation of wettest seasons is just sufficient to meet maximum demands of evaporation and transpiration. For San Luis Rey and San Diego Rivers, maximum seasonal precipitation is far too small to satisfy the demand, and the upper ends of the curves (at 45 in. of rainfall) have angles of departure with the 100% line. The condition necessary for maximum evapo-transpiration loss to occur from these water-sheds is greater annual precipitation and a regular succession of storms during the wet season at intervals just necessary to keep the soil at field capacity. On Fig. 15, the five rainfall-run-off curves have been assembled,

TABLE 18.—WATER-SHED CHARACTERISTICS FOR TWO OREGON AND THREE SOUTHERN CALIFORNIA STREAMS.

TABLE 18.—WATER-SHED CHARACTERISTICS FOR TWO OREGON AND THREE SOUTHERN CALIFORNIA STREAMS.

Item No.	Name.	South Fork, Coquille River, above Powers, Ore.	Rogue River, above Raygold, Ore.	Devil's Canyon Creek, above gauging station, near San Bernardino, Calif.	San Luis Rey River, above Pala, Calif.	San Diego River, above "Diverting Dam," Calif.
15	Maximum evapo-transpiration loss, in inches	32.5	21.0	37.6	40.0±	46.5±
1	Elevation above sea level, in feet: Mean	.....	.....	3 600	.....	.....
2	Elevation above sea level, in feet: Maximum	3 200	6 500	5 250	5 250	4 500
3	Elevation above sea level, in feet: Minimum	220	1 100	1 750	650	750
4	Area, in square miles	168	2 020	6.16	322	90
5	Annual seasonal precipitation: Character	Rain with occasional snow at higher levels.	Snow with rain at lower levels.	Rain with occasional snow at higher levels.	Rain with occasional snow at higher levels.	Rain with occasional snow at higher levels.
6	Amount, in inches	96.0	43.6	34.3	25.6	27.8
7	Seasonal distribution; 75% occurs in the period:	November 1 to March 31.	November 1 to April 30.	December 1 to March 31.	November 1 to March 31.	November 1 to March 31.
	Temperature, in degrees Fahrenheit:					
8	Mean annual	52.0	48.0	.....	58.6	56.2
9	Maximum monthly	62.9	66.6	.....	73.7	69.3
10	Minimum monthly	41.4	31.4	.....	44.2	44.2
16	Average annual evaporation from large free water surface, in inches	36±	45±	55±	62.2	71.0
11	Topography	Mostly steep slopes, low relief in flats, and ridge tops at head of drainage.	Moderate slopes in mountains and foothills; flat slopes in valleys (8 per cent.).	Steep slopes.	Largely steep slopes with central portion occupied by large low relief valley (15 per cent.).	Mostly steep slopes, small low relief valleys at head of drainage.
12	Soil	Shallow residual with much humus and forest litter.	Decomposed lava and pumice in mountains; shallow residual in foothills; alluvial in valley.	Shallow residual with decomposed granite subsoil.	Shallow residual with decomposed granite subsoil, clayey soils in valley.	Shallow residual with decomposed granite subsoil.
13	Geological formation	Early tertiary sandstone, shale, and conglomerate.	Mostly lava with crystalline rocks and shallow valley fill in lower portion.	Granitic with some old alluvium flats and detrital slopes.	Granitic with lacustral deposits occupying valley portion.	Granitic.
14	Vegetation: Character and density during the growing season	Dense forest cover of coniferous and deciduous trees with leafy underbrush.	Coniferous forest cover in mountainous portion, foothills and valley, brushy or bare.	Dense brush cover with alder and sycamore in stream beds.	Dense to thin cover on slopes with oak and pine at higher levels, valley margin and floor bare.	Dense brush cover with sycamore and willows in stream bed.
17	Evaporation opportunity	0.90	0.47	0.68	0.64	0.66

and with the assumption of maximum conditions, the San Luis Rey and San Diego curves have been extended in conformity with the trends of the upper ends of the supported portion of the curves and with a general shape similar to the Devil's Canyon curve. The maximum annual evapo-transpiration loss for each of the five streams as scaled from Fig. 15, together with the annual evaporation from a large free water surface and computed evaporation opportunity ratio, are listed as Items Nos. 15, 16, and 17, respectively, in Table 18.

"Evaporation opportunity" is a very useful coefficient because it represents quantitatively the ratio of maximum evapo-transpiration loss for a given combination of meteorological and physical conditions to the evaporation from a large free water surface for the same meteorological conditions. Analyzing the values in Items Nos. 15, 16, and 17 of Table 18, the Coquille, with a coefficient of 0.90, shows the combined effects of maximum interception of rainfall by dense vegetation, the maximum holding of moisture within the root zone by vegetation mat and humus soil, and unrestrained transpiration from a 100% vegetation cover. The Rogue, with a coefficient of but 0.47, registers the effectiveness of lava in absorbing moisture from melting snow and transmitting it downward below the root zone, and also the more open character of the forest cover and the sparseness of vegetation in the valley area. The three Southern California water-sheds have physical characteristics quite similar to the Coquille, with the exception of vegetation type and density. The coefficients average 0.66. This value is less than for the Coquille principally because of difference in the extent of interception by vegetation, and the difference in transpiration loss, the latter due both to differences in rate and in density of vegetation.

The analysis and comparison of the complete curves for these five streams suggest a possible method for logical construction of run-off curves for mountain water-sheds for which run-off data are incomplete. The control for constructing such curves is comprised largely in determining values for: (1) The minimum rainfall necessary to produce run-off from the water-shed; (2) evaporation opportunity; (3) annual evaporation from large free-water surface for the meteorological conditions of the water-shed; (4) the vegetational tolerance with respect to variations in water supply; and (5) the storm distribution of seasonal rainfall.

(1) The minimum rainfall necessary to produce run-off from the water-shed determines the point of beginning of the curve, and for mountain water-sheds it varies from 5 to 10 in., depending upon water-shed characteristics and the index of wetness.

(2) The ratio, evaporation opportunity, varies for different water-sheds from 0.40 to slightly more than 1.0, depending largely upon the physical conditions and vegetation cover. If the annual evaporation from a large free water surface and the evaporation opportunity are known, the maximum evapo-transpiration loss can be computed. This, in turn, makes it possible to establish the position of the straight-line section of the run-off curve with respect to the 100% run-off line.

(3) Annual evaporation should be as observed in a floating submerged pan exposed on a large free water surface, such as a lake or reservoir. Its ordinary range of variation is from 30 to 72 in.

(4) Vegetational tolerance with respect to variations in water supply, has considerable influence on the shape and location of the point of tangency of the straight line and the curved part of the run-off curve. The more tolerant the vegetation, the flatter the curve and the greater the depth of run-off at which the change from curve to straight line occurs. Deciduous and coniferous trees are normally very exacting in their water requirements. Brush cover is quite tolerant. The most flexible types of vegetation are the annuals, such as flowers, weeds, grasses, tules, etc.

(5) The storm distribution of seasonal rainfall has a direct bearing on total loss by interception and soil evaporation. Maximum conditions occur for a regular sequence of storms throughout the wet season at intervals of proper length to keep soil moisture at field capacity. Its relation to the run-off curve corresponds with that of vegetational tolerance.

The principles which govern valley run-off plus deep percolation are essentially the same as for mountain water-sheds, and it would appear that the foregoing method could be extended to valley water-sheds with appropriate modifications. The item of greatest uncertainty, because of the many factors influencing it and the lack of digested data, is the evaporation opportunity. Factors which would tend to make this ratio larger for valley than for mountain water-sheds are:

(*a*) The usual absence of shallow sloping bed-rock in valleys, which in mountain water-sheds tends to hold percolating water within the root zone of vegetation.*

(*b*) The greater intensity of storms in the valley, at least in Southern California.*

(*c*) Lower density of natural vegetation cover in a valley, which affects both interception of rainfall and the transpiration aggregate. The effect of Factor (*c*) may be somewhat offset by increased exposure of the soil to evaporation.

Factors that tend to make the ratio larger are: (1) Flat topography in valleys instead of broken, as on water-sheds; (2) cultivation of valley lands; and (3) in the valley, greater depth below the ground surface to which moisture depletion less than field capacity may occur.

Referring to Factor (1), the actual land area of a mountain water-shed may exceed the projected map area by from 10 to 20 per cent. If the conversion of measured run-off from volume to depth is made on the basis of actual land area, the result will be correspondingly reduced. This may result in an increase of evaporation opportunity for valley water-sheds of from 5 to 10 per cent. Cultivation of valley lands (Factor (2)) offers greater opportunity for absorption by the soil during storms and thus greater loss by soil evaporation between storms.

In the absence of actual data for the evaporation opportunity ratio for valley water-sheds, assumptions can be made based upon the ratio in adjacent mountain water-sheds with modification as comparison of influencing factors

* *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, pp. 1155-1158.

suggests. A run-off curve may then be constructed following the outline described herein and using values for other items which ordinarily are available. The resulting run-off curve includes deep percolation and storm run-off. The former can then be segregated by constructing a storm-water run-off curve from measurements of valley run-off, and taking off of differences.

The writer has applied this method to the determination of penetration in mountain water-sheds, and because of its practical usefulness in constructing mountain run-off curves, he believes that its use may be extended as future determinations of evaporation opportunity for type valley water-sheds are made.

Determination of Rainfall Penetration from Test Wells in Pauba Valley.—The method of determining rainfall penetration from test wells in Pauba Valley, as described by the author,* has distinct possibilities where it can be applied free from influences other than precipitation. During the winter of 1913-14 and 1914-15 the writer had under observation more than 120 wells and test holes in various valleys of San Diego County, California, many of which were located in valleys of a character similar to the Pauba Valley. This experience led to caution in the interpretation of water-level observations under the conditions such as exist in the Pauba Valley. One of the sources of difficulty was the influence of flood run-off in an adjacent stream upon water level in test holes in the same alluvial formation. This interference may occur either as a water-table "wave" traveling slowly outward from the stream channel, or as a quick transfer of pressure through a stratum of coarse sand. The former would not produce sudden rises, such as are indicated on Fig. 5,† unless test wells were located close to the channel. It is possible, however, that such rises were the result of pressure transfer not reflected in the water-table. In the case of the three wells shown on Fig. 5, the possibility of a pressure effect could be easily determined by ascertaining whether flood run-off in Temecula River occurred during the storm of December 3 to 10, 1927. During this storm nearly 9 in. of rain fell and there was no sudden rise in water level in the wells, due to the moisture deficiency of the sand.

The author omits to state his method of determining the factor used in converting observed rise of water-table to equivalent depth of water. He states that "a porosity factor of 25% was allowed".‡ The writer has frequently had occasion to make this evaluation, and has developed a graphical method which takes into account existing capillary moisture above the water-table before rise commences. Apparently, the author has not given attention to this phase.

The method is based on the fact that the volume of water in a soil column of given depth and unit area is equal to the average percentage of moisture by volume in the column multiplied by the depth. Hence, on a moisture percentage-depth graph (see Fig. 17), the volume of moisture in any column is represented graphically by the area between the moisture curve and the line of zero moisture. The volume of water represented by a given rise in the water-table, is equal to the difference of the water in the soil between the original

* *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1162.
† *Loc. cit.*, p. 1149.
‡ *Loc. cit.*, p. 1163.

water level and the surface of the ground, after and before the rise occurred. This volume is graphically represented by the area between the moisture percentage curves before and after the rise. For ordinary conditions these curves are similar. The following is the development of a simple equation for computing the volume. Since a soil column of unit area is assumed, volume and depth have the same value. For a soil column 1 sq. ft. in cross-section, extending from the ground surface to the lowest level of the water-table (Fig. 17):

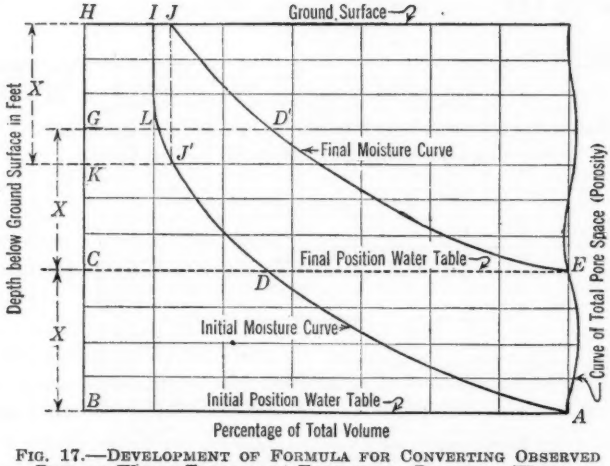


FIG. 17.—DEVELOPMENT OF FORMULA FOR CONVERTING OBSERVED RISE OF WATER-TABLE INTO EQUIVALENT DEPTH OF WATER.

Let v = volume or depth of water represented by a rise, X, of water-table;

r = volume or depth of water represented by the total pore space of soil through which water-table rises; and

c = volume or depth of water in the soil between the ground surface and a level, X ft. below the ground surface.

Then, because of geometric relations,

$$v = \text{Area } ADE + \text{Area } EDLIJD'$$

$$r = \text{Area } ADE + \text{Area } ABCD = \text{Area } ADE + \text{Area } ECGD'$$

$$c = \text{Area } J'KHI = \text{Area } HILG + \text{Area } GLJ'K = \text{Area } HILG + \text{Area } DCGL - \text{Area } DCKJ'$$

Hence,

$$v = r - \text{Area } ECGD' + \text{Area } EDLIJD'$$

but,

$$\text{Area } EDLIJD' = \text{Area } ECGD' - \text{Area } DCGL + \text{Area } D'GHJ - \text{Area } HILG$$

Substituting,

$$v = r - \text{Area } DCGL + \text{Area } D'GHJ - \text{Area } HILG$$

and since,

$$\text{Area } D'GHJ = DCKJ'$$

therefore,

$$v = r - c \qquad (2)$$

In other words, the volume of water represented by a rise of X ft. in the water-table equals the total pore space in the soil column through which the water-table rises, minus the moisture in the soil column between the ground surface and a level X ft. below the ground surface.

In the use of Fig. 17, the average percentages for total porosity in the column X ft. above the initial position of the water-table, minus the average percentage of moisture on the column X ft. below the ground surface, multiplied by the rise in the water-table, in feet, gives the equivalent depth of actual water, in feet.

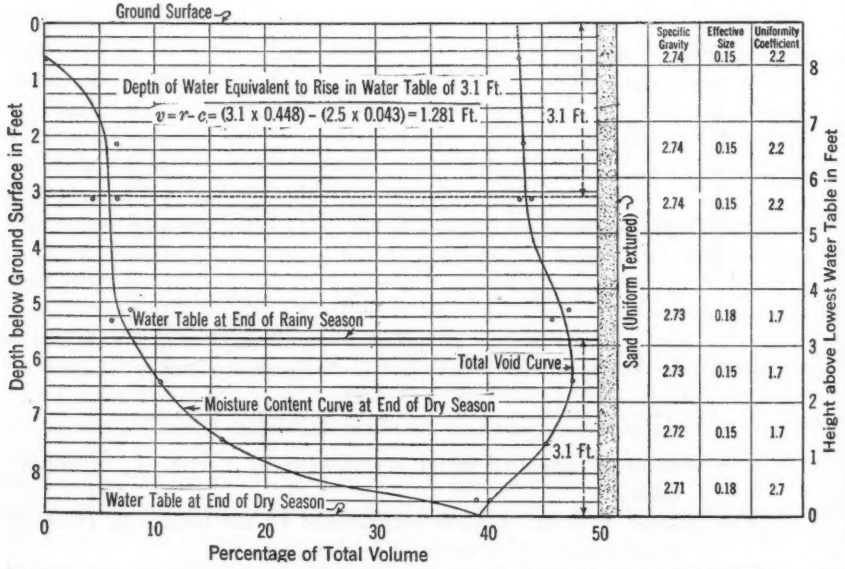

Ground Surface
Depth of Water Equivalent to Rise in Water Table of 3.1 Ft.
$v = r - c = (3.1 \times 0.448) - (2.5 \times 0.043) = 1.281$ Ft.
3.1 Ft.
Water Table at End of Rainy Season
Total Void Curve
Moisture Content Curve at End of Dry Season
3.1 Ft.
Water Table at End of Dry Season
Depth below Ground Surface in Feet
Height above Lowest Water Table in Feet
Percentage of Total Volume
Sand (Uniform Textured)
Specific Gravity 2.74
Effective Size 0.15
Uniformity Coefficient 2.2
2.74 0.15 2.2
2.74 0.15 2.2
2.73 0.18 1.7
2.73 0.15 1.7
2.72 0.15 1.7
2.71 0.18 2.7

FIG. 18.—APPLICATION OF FORMULA FOR CONVERTING OBSERVED RISE OF WATER-TABLE INTO EQUIVALENT DEPTH OF WATER.

In applying the method, type areas are selected such that the character of the soil and subsoil is more or less uniform. In each area a pit is dug to the water-table at the end of the dry season. Samples are cut from the exposed face, at intervals of 1 ft., or more, so as to be representative of the gradations of material in the column, and initial moisture and total porosity are determined. If desired, a mechanical analysis of each sample may also be run, although this is not necessary. The data are then prepared in graphical form, as illustrated by Fig. 18, which represents the condition where the depth to water-table exceeds the capillary limit of the material. This diagram represents a typical material for similar valleys in this region, including Pauba Valley. It is to be noted that the computed depth of equivalent water represents percentage by volume of 39.4 for a material with an average porosity of 45 per cent. It is probable that the author's assumption of 25% for net porosity* is too low.

* *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1163.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

SILT TRANSPORTATION BY SACRAMENTO AND COLORADO RIVERS AND BY THE IMPERIAL CANAL

Discussion*

By Ivan E. Houk, M. Am. Soc. C. E.

Ivan E. Houk,† M. Am. Soc. C. E. (by letter).‡—The author has contributed much of value to the subject of silt transportation. He has also presented some interesting discussions of miscellaneous river phenomena, such as boils, pulsations, sand waves, etc. The conditions which cause the formation and occurrence of sand waves are fairly well understood, but those resulting in boils and pulsations are more or less uncertain, despite the fact that such phenomena occur in practically all conditions of flowing water.

Sand waves are known to result from the movement of material on the stream bed. Their action can be studied intensively in the larger sluicing works at canal intakes such as have been built on many irrigation and power projects of Western United States. In the settling basin at the Fort Laramie Canal intake, North Platte Irrigation Project, in Wyoming, sand waves frequently form during sluicing operations.§ Sometimes they form at the outlet end of the basin and move up stream as fast as a man can walk. (See Figs. 7 and 8.)

In 1914 the writer made some investigations of boils and pulsations in the flow of the Miami and Erie Canal at Dayton, Ohio. The section of the canal investigated was about 50 ft. wide and about 3 ft. deep; it had a slope of approximately ½ ft. per mile, and usually carried from 100 to 150 sec-ft. of slightly turbid flow at average velocities of less than 1 ft. per sec. No measurable proportions of sand or silt were transported and no movement of material occurred on the stream bed. The investigations were made in connec-

* Discussion of the paper by C. E. Grunsky, Past-President, Am. Soc. C. E., continued from February, 1930, *Proceedings*.

† Senior Engr., U. S. Bureau of Reclamation, Denver, Colo.

‡ Received by the Secretary, January 31, 1930.

§ "Sand Control Works at Fort Laramie Canal Intake," by Ivan E. Houk, M. Am. Soc. C. E., *Engineering News-Record*, June 14, 1928, p. 922.

tion with the roughness factor experiments described in the Miami Conservancy District Technical Reports and a detailed description of the location was included in that publication.* However, the results of the investigations of boils and pulsations were not published.

Although the quantities of water flowing in the canal were relatively small, and the velocities relatively low, the conditions of flow were quite irregular. At times, the water surface would be covered with boils of varying magnitude; at other times, it would appear to be perfectly smooth and uniform. Sometimes, the surface would be so smooth that the entire body of water would appear to stop and remain stationary for a few seconds, and then to move on down stream in a solid mass with only minor surface disturbances. On other occasions the current would shift from one side to the other in a very noticeable manner. Although in a few cases the water would seem to boil up from the bottom and spread out on one side, or on all sides, without any apparent regularity, in the majority of cases there was a definite rotary movement. The direction of rotation was practically always contra-clockwise on the left side of the channel, and contra-clockwise in the majority of cases on the right. Although theoretically, this direction of rotation would be caused by the rotation of the earth, it is much more likely that the conditions that caused the formation of the boils also caused the direction of rotation, except for such effects as would be produced by the velocity distribution in the canal. Since the velocities increased from the sides of the channel toward the center, the velocity distribution would tend to cause a contra-clockwise rotation on the left side of the canal.

The investigations of boils and pulsations were made for the purpose of determining what, if any, relation existed between these two phenomena; whether they had any effect on the accuracy of current meter gaugings of discharge; and whether their formation and occurrence were in any way related to the growth of moss on the bed and sides of the channel. Observations consisted of careful, prolonged, current meter measurements of velocity, two-tenths and eight-tenths of the depths, at certain stations in the cross-section, for the purpose of determining velocity pulsations. Notes were taken regarding size, frequency, location, direction of rotation, and general characteristics of boils. Readings were taken on September 15, October 6 and 9, and November 6, 1914. On the first date, the channel was in ordinary mid-summer condition, weeds and grass lining the banks and moss covering the bottom in many places. On the second and third dates, the sides and bottom of the channel had been thoroughly cleaned of weeds, grass, moss, and débris through a distance of about 50 ft. extending up stream from the gauging station; and both banks had been cleaned to some extent for several blocks in both directions. On the last date mentioned, the sides and bottom had been very carefully cleaned of all obstructions for about ¼ mile in each direction.

No connection whatever could be observed between the boils noticeable on the surface and the velocity pulsations indicated by the current meter; the variations in velocity at times were greatest when the surface appeared to be

* "Calculation of Flow in Open Channels," by Ivan E. Houk, M. Am. Soc. C. E., Miami Conservancy Dist. Technical Repts., Pt. IV, Dayton, Ohio, 1918, p. 111.

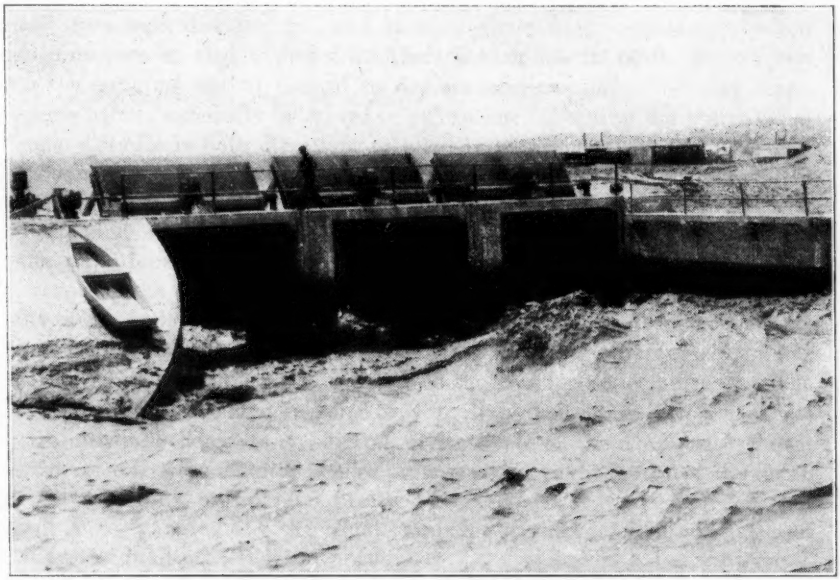


FIG. 7.—VIEW OF TYPICAL SAND WAVES, FORT LARAMIE BASIN, DURING SLUICING PERIOD WITH VELOCITIES VARYING FROM 10 TO 15 FEET PER SECOND.

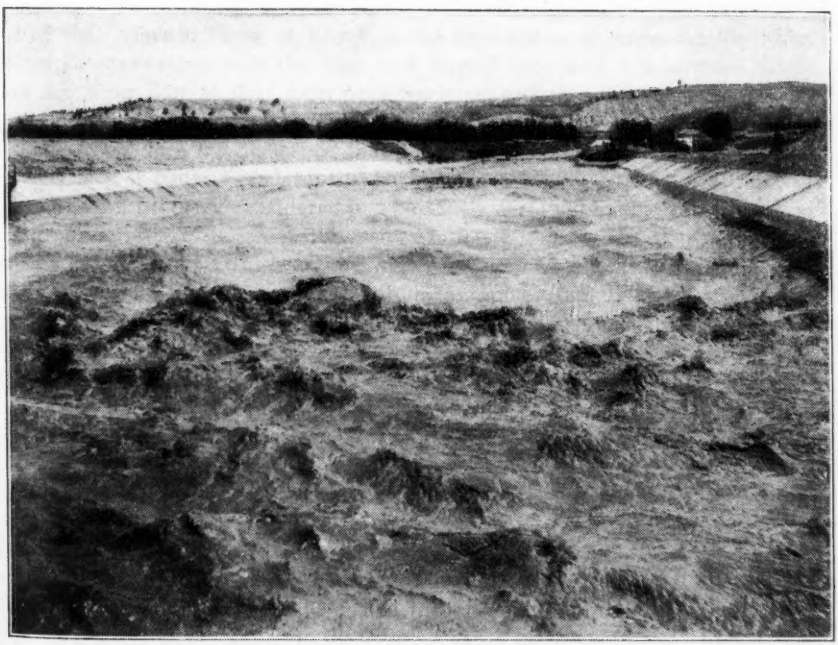


FIG. 8.—SAND WAVES IN SETTLING BASIN AT FORT LARAMIE CANAL INTAKE, DURING SLUICING OPERATIONS.

entirely
the sur
above
they we
for son
were o
more i
the we
up the
The ca
not lea
to flow
wheels
boils p
channe
tions i
than a
curren
much

Th
and C
sidera
load o
studie
from
to 30%

Th
bugab
bed lo
of silt
suspe
taken
sampl
seque
inclu

Th
is not
into
comp
consi
appre
capab
of th
river
loads

entirely free from disturbances, and at other times they were greatest when the surface was covered with boils. The cleaning of the 50-ft. section just above the gauging station seemed to make conditions definitely worse than they were before, especially in regard to pulsations. Cleaning the entire canal for some distance in both directions resulted in better conditions of flow than were observed on the second and third dates. However, the flow was still more irregular than it had been before any cleaning was done. Apparently, the weeds and moss acted somewhat like baffles in a settling basin, breaking up the pulsations instead of creating them as had been thought possible. The cause of the formation and occurrence of the boils and pulsations was not learned. It is quite likely that the pulsations were related in some way to flow conditions at points up stream, such as the discharge from water wheels at plants along the canal which used the flow for power purposes. The boils probably were related in some way to irregularities in the bed of the channel although there was no shifting of materials on the canal bed. Pulsations in velocity were definitely more pronounced at eight-tenths of the depth than at two-tenths, as would naturally be expected. It was concluded that current meter gaugings by the usual two-point method might be in error as much as 4% due to the effect of pulsations.

The author's estimates of quantities carried as bed loads in Imperial Canal and Colorado River,* although naturally approximate, are well worth consideration. The estimate of from 25 000 to 30 000 acre-ft. as the annual bed load of the Colorado River at Yuma, Ariz., does not seem unreasonable when studied in connection with the long and careful record of silt samples taken from the river flow at that location. Such an estimate is approximately 25 to 30% of the total average annual silt load indicated by the samples.

The writer does not feel sure that the bed-load problem is as much of a bugaboo as it is usually made out to be; or that a large part, if not all, of the bed load is not properly considered in a carefully designed and regulated system of silt sampling. It is generally assumed that silt sampling determines the suspended load only. However, the writer is inclined to believe that samples taken at, or just above, the river bottom, as approximately one-third of the samples were at Yuma, include a large proportion of bed-load material. Consequently, estimates of annual silt load based on such a record would tend to include a large part of the so-called bed load.

The writer's idea of bed load in the case of a river like the Lower Colorado is not that it is a solid mass of moving material at the bottom of a channel, into which a silt sampler will refuse to penetrate; but, rather, that it is a comparatively thick mixture of sediment and water of gradually increasing consistency, close to the bed of the stream, which is easily penetrated to appreciable depths by a comparatively heavy sampler and, consequently, is capable of being sampled. Of course, accurate samples of flow at the bottom of the channel could not be expected in the case of extremely large floods in a river like the Colorado; or in the case of large streams transporting heavy loads of gravel, cobbles, or boulders by saltation, unless unusually large sums

* *Proceedings*, Am. Soc. C. E., August, 1929, Papers and Discussions, pp. 1487–1488.

of money were available for such investigations. However, there undoubtedly are many silt-bearing streams of characteristics such that a carefully regulated and relatively inexpensive system of sampling would determine the total silt load. This would be especially true of artificial channels, such as irrigation canals.

The fact that fairly reliable predictions of reservoir silting can be made from samples of river flow has already been demonstrated. About 1913, the late W. W. Follett, M. Am. Soc. C. E., estimated the average silt load of the Rio Grande at San Marcial, N. Mex., for the period from 1897 to 1912, inclusive, at 19 739 acre-ft.* In 1916, storage of water was begun in Elephant Butte Reservoir, just below San Marcial; and since that time rates of silt deposition within the reservoir have been determined by hydrographic and topographic surveys made at about 5-year intervals. A study of the results shown by the reservoir measurements was made after the 1925 survey and, later, R. F. Walter, M. Am. Soc. C. E., showed that the average annual rate of silt deposition within the Elephant Butte Reservoir, for the period from 1916 to 1925, was 20 508 acre-ft., or only 4% greater than Mr. Follett's estimate.† The latter estimates showed an average silt load equal to 1.66% of the total run-off, whereas the U. S. Bureau of Reclamation's reservoir and stream flow measurements showed an average annual silt deposition equal to 1.64% of the total run-off.

Silt entering the upper end of Elephant Butte Reservoir, although relatively fine, is coarser and heavier than silt in the Colorado River at Yuma. Consequently, the proportion of total silt load carried as bed load probably is greater in the Rio Grande than in the Colorado. The total annual flow in the Rio Grande is much less than in Colorado, floods are much smaller, and bed-cutting during flood periods is much less pronounced.

Mr. Follett's estimate of average annual silt load at San Marcial was based on a 16-year record of stream flow and silt sampling, the first half of which was computed from observations at El Paso, Tex., proper corrections being applied to allow for differences in discharge at the two locations. Only a limited number of samples were available for determining the silt content of the river during the first 8-year period; but a total of 824 were available for the second, which was an average of 103 samples a year for the last 8 years of the 16-year period. Mr. Follett's estimates may have contained compensating errors which would not necessarily exist in other cases. He assumed the average unit weight of dry Rio Grande sediment to be 53 lb. per cu. ft., a figure obtained by securing one 3-in. cube of silt and determining its dry weight. The cube was carefully cut from a sediment bar just above the Mexican Dam, at El Paso, on June 29, 1904, after the spring flood had subsided and after considerable shrinkage had taken place in the silt deposit. The Bureau of Reclamation's subsequent investigations indicate an average weight of 85 lb. per cu. ft. for dry Rio Grande silt. Moreover, it is understood that after Mr. Follett's *Bulletin* was published an error was found in

* "Silt in the Rio Grande," by W. W. Follett, *Bulletin*, U. S. Dept. of State.
† *Transactions*, Am. Soc. C. E., Vol. 93 (1929), p. 1727.

his original computations, and that the correction of this error would have increased his value of 53 lb. per cu. ft.

If it is assumed that Mr. Follett's estimates of silt load volume should have been based on a unit weight of 85 lb. per cu. ft. instead of 53 lb., his estimate of average annual silt load at San Marcial would have been 12 308 acre-ft. instead of 19 739, or only 60% of the Bureau of Reclamation's measured value. Assuming the value obtained from the reservoir surveys to be correct, the estimate made from the samples of river flow, which, in this case, would be mostly suspended sediment, would leave 8 200 acre-ft., or 40% of the true total, as a possible average annual bed load in the Rio Grande. It thus appears that Mr. Follett's use of a relatively low unit weight of silt may have approximately offset his inability to secure accurate quantitative data on bed-load transportation. However, be that as it may, the interesting fact remains that his estimate of average annual silt load was almost identical with the value obtained by careful reservoir surveys.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

ANALYTICAL SOLUTION OF MASONRY DOMES: UNBALANCED LOADS

Discussion*

By Messrs. A. Floris and David C. Coyle†

A. Floris,‡ Esq. (by letter).§—It is rather doubtful whether a general theory of unsymmetrically loaded domes is much needed in a practical design of these structures. In domes the main loads are those due to their own weight and it is only in flat structures that the stresses produced by snow can be appreciable. Furthermore, domes which are relatively high in comparison with their span must be investigated also with regard to stresses caused by wind pressure. Any other loading condition is almost impossible in practice because domes, being mostly monumental structures, are symmetrical and symmetrically loaded, thus excluding in advance other possibilities. They possess not only symmetry of shape but, with the exception perhaps of wind pressure, symmetry of loading as well.

It is very fortunate that this is the case since the analysis of domes carrying eccentric loads arbitrarily chosen is not feasible to date. The theory of shells of rotation and especially of domes under axially symmetrical loading or wind pressure has received considerable attention abroad in recent years, and has been brought to a high degree of perfection. For reference, a bibliography of this subject is appended hereto. These investigations have revealed the important fact that the stress distribution in domes depends not only on their shape, but on the external loads as well, so that for any given load there is a stress variation corresponding to it.

These investigations have shown that the same law of stress distribution does not hold for various loading conditions. Navier's theory of bending as developed for prismatic bars cannot be assumed to apply in the case of domes. For this reason the author's fundamental assumption that the law

* Discussion on the paper by David C. Coyle, M. Am. Soc. C. E.
† Author's closure.
‡ Civ. Engr., Los Angeles, Calif.
§ Received by the Secretary, January 7, 1930.

of the trapezoid will be valid in the case of unsymmetrically loaded domes, which are systems in space, is more than a rough approximation; it is absolutely impossible to apply. Consequently, formulas derived under this assumption are of doubtful value in the design of domes and similar structures.

In addition, the author treats the analysis of wind stresses in a dome under the same assumption of the planar theory of flexure. The problem of wind stresses in domes has been solved by other writers for a certain distribution of wind pressure only. Assuming a pressure on the windward, and a suction on the leeward, sides of the dome, the stresses thus produced can be analyzed if the external loads are expressible in a Fourier series. They can be determined either by solving the differential equations of the problem, or, more simply, by statics, as has been shown recently by F. Dischinger.* This special wind loading with pressure on one and equal suction on the other diametrically opposed sides of the dome is sometimes called "anti-symmetrical".

Theory of Stress Distribution in Beams with Anti-Symmetrical Wind Pressure.—The thickness of a dome is usually small as compared with the dimensions of rise and span, so that it can be considered as a shell generated about an axis of rotation. In such a shell the stresses due to the external loads are, under certain conditions, membrane stresses, that is, they are axial, acting in its mid-surface so that they can be determined by statics. This stress analysis for axially symmetrical loads was first given by Lamé and Clapeyron† more than one hundred years ago and since then it has been repeatedly used by many writers. The stresses due to the boundaries (support, change of curvature, etc.) of the shell are caused mainly by bending moments and must be determined by taking into account the deformation of the structure. They are statically indeterminate and, at the same time, are independent of the external loads.

In a shell of rotation, with an axially symmetrical loading, the meridian and ring stresses are membrane stresses and, since they are normal to the corresponding sections of the shell, they are principal stresses. In an anti-symmetrically loaded shell of rotation, on the other hand, the meridian and ring stresses, although they are membrane stresses, do not act normal to the sections of the shell, but are inclined. By taking the components of these stresses, shearing stresses acting in the mid-surface of the shell are produced, in addition to the normal (meridian and ring) stresses. In the case of an anti-symmetrical loading the stresses due to the boundaries of the shell differ from those caused by an axially symmetrical loading.‡ However, they can be determined approximately by using the methods applied to the axially symmetrical loading.§

Neglecting the stresses due to the boundaries of the shell the rigorous analysis of the membrane stresses in a spherical shell has been given by

* "Schalen und Rippenkuppeln," von Franz Dischinger, Handbuch für Eisenbetonbau, Vol. 6, Pt. 2, Fourth Edition, Berlin, 1928, p. 197.

† In "Mémoires présentées par divers savants," No. 4, 1828, p. 465.

‡ "Ueber Spannungen in symmetrisch und unsymmetrisch belasteten Kugelschalen (Kuppeln), insbesondere bei Belastung durch Winddruck," von E. Schwerin, Thesis, Berlin, 1918; also, abbreviated in *Armierter Beton*, No. 12, 1919, p. 25.

§ "Schalen und Rippenkuppeln," p. 247.

H. Reissner* in the case of loads expressible in a Fourier series. He has determined these stresses for a wind pressure following the sine law. Later, Dischinger† discovered that, although this pressure distribution which is commonly assumed in design, is greater than the square sine law, nevertheless, the stresses produced by the former were found to be smaller than those due to the latter. This is true, however, only in the spherical shells. In shells with a polygonal base the greatest wind pressure produces also the greatest stresses, especially meridian stresses. These shells with polygonal rings but with curved meridians are a new type of dome possessing very favorable and, at the same time, interesting properties.

The important result in Dischinger's investigations as applied to the paper under discussion is the fact that only for the sine law of the wind pressure are the stresses in a spherical shell linearly distributed.‡ In this case alone is there a neutral axis of the meridian—and ring—stresses, which is perpendicular to the wind direction. (See Fig. 10 (*a*).) The intensity of the

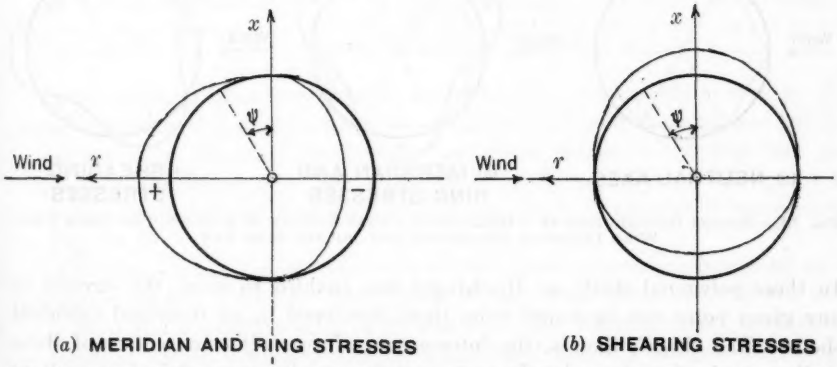


FIG. 10.—STRESS DISTRIBUTION IN A HORIZONTAL CROSS-SECTION OF A SPHERICAL DOME FOR A WIND PRESSURE FOLLOWING THE SINE LAW.

shearing stresses, on the other hand, reaches its maximum in the neutral axis disappearing in the points where the normal stresses have their greatest values (see Fig. 10 (*b*).) In all other cases of anti-symmetrical wind pressure that follow various sine laws, the spherical shell possesses several neutral axes which not only do pass through the center of the parallel circles, but also connect the points of intersection of these axes with the circumference. (See Fig. 11 (*a*).) In this case, therefore, the distribution of the meridian and ring stresses is no longer linear. (See Fig. 11 (*b*).) The shearing stresses have their maximum values in the zero points of the normal (meridian and ring) stresses and *vice versa*. (See Fig. 11 (*c*).) Thus, for the various laws of wind-pressure distribution there are a certain number of corresponding

* "Spannungen in Kugelschalen (Kuppeln)," von H. Reissner, in Müller-Breslau Festschrift, Leipzig, 1912.

† "Schalen und Rippenkuppeln," p. 200.

‡ *Loc. cit.*, p. 198.

neutral axes or points of change in the signs of stresses. For a few cases, these are as follows:

Wind pressure variation.	Neutral axes.
sin ψ	$1 = 1$
sin 3ψ	$3^2 = 9$
sin 5ψ	$5^2 = 25$
....	.....
sin $n\psi$	n^2

In this list sin ψ corresponds to the sine law of wind pressure, while sin 3ψ expresses the square sine law of this pressure*.

The author apparently does not realize the great importance of the pyramoidal domes, as he calls those with a curved meridian and a polygonal base. Thus, he treats an entirely new type of dome unaware of its remarkable properties with their far-reaching consequences in structural engineering.

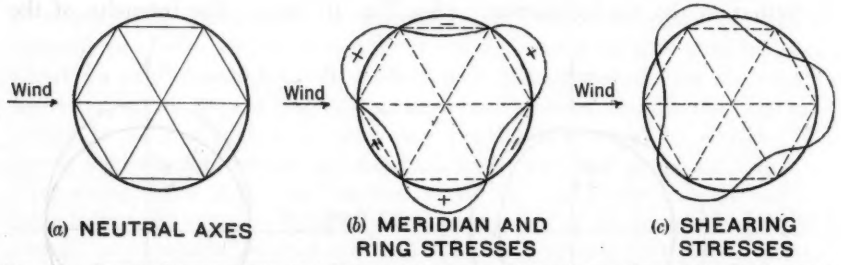


FIG. 11.—STRESS DISTRIBUTION IN A HORIZONTAL CROSS-SECTION OF A SPHERICAL DOME FOR A WIND PRESSURE FOLLOWING THE SQUARE SINE LAW.

In these polygonal shells, as Dischinger was enabled to show, the stresses in any given point can be found from those developed in an inscribed spherical shell.† The "hips", that is, the intersection of two adjacent sectors of these shells, remain free from bending stresses for axially symmetrical as well as anti-symmetrical loadings, provided the structure possesses a symmetry of shape. While in the shells of rotation the loads are transmitted to the abutment by the aid of the meridian stresses, the loads in the polygonal shells are sustained partly by the shell itself and partly by its "hips". In the polygonal shells, therefore, there are developed purely membrane stresses, a fact which is very important from the point of view of economy. These shells which are stressed mainly axially can have very thin sections because their weight does not increase appreciably with increasing span, as is the case, for instance, in rigid frames and similar structures. This discovery combined with that of the self-supporting arch barrel‡ is the latest successful application of the advanced theory of statics to the problems of structural engineering.

* "Schalen und Rippenkuppeln," pp. 200 and 208.

† "Die Theorie der Viereckkuppeln und die Zusammenhänge mit den einbeschriebenen Rotationsschalen," von Franz Dischinger, *Beton und Eisen*, 1929, No. 5, pp. 100-107; No. 6, pp. 119-122; No. 8, pp. 150-156; and No. 9, pp. 169-175.

‡ "Schalen und Rippenkuppeln," p. 269; "Die Schalendächer des Elektrizitätswerkes in Frankfurt a. M.," von U. Finsterwalder, *Beton und Eisen*, 1928, pp. 205-209; "Die Schalenbauweise Zeis-Dywidag," von Fr. Dischinger und U. Finsterwalder, *Bauingenieur*, 1928, Nos. 44 to 46; also, "Self-Supporting Arch-Barrel," by A. Floris, *Western Construction News*, April 10, 1929.

With regard to the stress distribution of the polygonal shells the results of the theory developed by Dischinger for the anti-symmetrical wind pressure following the sine law will be sketched briefly. Owing to this law of pressure variation these shells have but one neutral axis.

The meridian stresses within a sector between two hips are constant and vary only from sector to sector according to the sin ψ law (see Fig. 12 (*a*)). Consequently, by assuming that the resultants of the meridian stresses are acting in the middle of these sectors, the same law of stress variation is valid as in the shell of rotation.

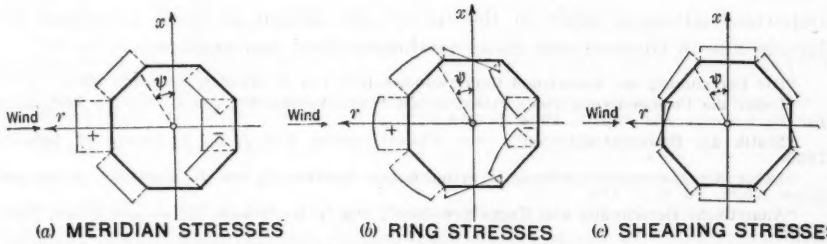


FIG. 12.—STRESS DISTRIBUTION IN A HORIZONTAL CROSS-SECTION OF A PYRAMOIDAL DOME FOR A WIND PRESSURE FOLLOWING THE SINE LAW.

Two kinds of shearing stresses are produced, namely, those due to the beam action of the shell between two adjacent hips and those caused by the dome action of the shell itself. The maximum values in the first case are reached in the sector $\psi = 90°$, while those of the second case take place at $\psi = 0$. The combined diagram of these two kinds of stresses is represented in Fig. 12 (*b*).

The ring stresses in a sector due to the beam action are composed of a parabolically and a linearly distributed stress. Both these stresses are zero at $\psi = 0$ and maximum at $\psi = 90$ degrees. The linearly distributed stress is constant within a sector. On the other hand the ring stresses due to the dome action of the shell, which are similar to those of a spherical shell, vary from hip to hip according to the sin ψ law. However, the difference in this case will be that the ring stresses remain constant within a sector and have as in spherical shells a maximum value at $\psi = 90°$, while they vanish at $\psi = 0$. The addition of the ring stresses is shown in Fig. 12 (*c*). With an increasing number of sides in a polygonal shell the ring stresses approach, but remain always greater than, those of a shell of rotation.

The properties of the polygonal domes herein outlined are manifestly of an outstanding importance for the future development of reinforced concrete structures. Due to the combination of beam and dome actions in these shells the loads can be transmitted to the foundations by the aid of columns only, without the aid of intermediate supporting members, walls, beams, or arches, to a maximum distance of 130 ft. approximately between columns.

It is very doubtful therefore, whether domes designed with ribs subjected to bending can compare favorably with the shells of rotation or with those of a polygonal base which have to sustain largely axial stresses. The remarkable self-supporting ability of the polygonal shells is best illustrated by the

fact that each of the three reinforced concrete domes of the Great Market in Leipzig, Germany, has an inscribed diameter of 245 ft., a maximum thickness at the abutment 4¼ in., and weighs only 2 380 tons.*

In conclusion the writer wishes to express his regret for having criticized this practical and carefully prepared analysis of the unsymmetrically loaded domes. In doing so he intended merely to state known facts which have possibly escaped the attention of the author.

Bibliography of Studies on Domes.—The following bibliography of the subject is given for those interested. References to the few English, Italian, French, and American sources must purposely be omitted because the most important advances made in the theory and design of these structures are largely due to German and Swiss mathematicians and engineers.

"Die Berechnung der Kessel-und Gefässwandungen", von J. Kubler, Leipzig, 1902.

"Ueber die Formänderung eines zylindrischen Wasserbehälters", von C. Runge, *Zeitschrift für Mathematik und Physik*, 1904, p. 254.

"Statik der Baukonstruktionen", von Müller-Breslau, Vol. 2, Pt. 2, Article 16, Leipzig, 1925.

"Ueber die Spannungsverteilung in zylindrischen Behältern", von H. Reissner, *Beton und Eisen*, 1908, No. 6, p. 150.

"Analytische Berechnung von Kuppelgewölben", von O. Gottschalk, *Beton und Eisen*, 1908, p. 196.

"Die Berechnung ebener und gekrümmter Behälterböden", von P. Forchheimer, Berlin, 1909.

"Graphisches Verfahren für die Ermittlung der Spannungsverteilung in zylindrischen Behälterwänden", von K. Federhofer, *Beton und Eisen*, 1909, No. 16, p. 387.

"Achsensymmetrische Verzerrung in dünnwändigen Hohlzylindern", von R. Lorenz, *Zeitschrift* des Vereins deutscher Ingenieure, 1908, p. 1706. (With regard to unsymmetrical buckling, see *Physikalische Zeitschrift*, 1911, p. 225.)

"Berechnung gewölbter Platten", von H. Keller, Thesis, Zurich, 1912; pub. in *Mitteilungen über Forschungsarbeiten auf dem Gebiete des Ingenieurwesens*, No. 124, Berlin 1912; also, abbreviated in *Schweizerische Bauzeitung*, 1913, pp. 11, 122, 153, and 171.

"Die Berechnung der Spannungen in zylindrischen Behälterwänden mit veränderlichem Querschnitt", von Th. Pöschl, *Sitzungsberichte* der Kaiserl. Akademie der Wissenschaften in Wien, June, 1912.

"Zylindrische Hängeböden und Behälterwände", von Th. Pöschl, *Armierter Beton*, 1912, Nos. 5 to 6; also, *Zeitschrift* des Oesterreichischen Ingenieur- und Architektenvereins, 1912, Nos. 3 to 5.

"Berechnung von Behältern nach neueren analytischen und graphischen Methoden", von Th. Pöschl und K. von Terzaghi, Berlin, 1926.

"Experimentalle und theoretische Untersuchungen über die Festigkeit von Kegel- und Kugelböden", von E. Frankhauser, Thesis, Berlin, 1913; also, abbreviated "Die Festigkeit von Kegel- und Kugelförmigen Böden und Deckeln", *Zeitschrift* des Vereins deutscher Ingenieure, 1914, pp. 840 and 922.

"Asymptotische Integration", von O. Blumenthal, *Archiv für Mathematik und Physik*, 1912, p. 136.

"Ueber asymptotische Integration von Differentialgleichungen mit Anwendung auf die Berechnung von Spannungen in Kugelschalen", von O. Blumenthal, *Zeitschrift für Mathematik und Physik*, Vol. 62, No. 4, 1913, p. 343.

"Der kritische Aussendruck zylindrischer Röhre", von R. von Mises, *Zeitschrift* des Vereins deutscher Ingenieure, 1914, p. 750.

"Ausknicken von Schalen", von R. Zölly, Thesis, Zurich, 1915; see, also, "Drang und Zwang", von A. Föppl und L. Föppl, Munich, 1920, p. 388.

"Das Elastizitätsproblem für dünne Schalen von Ringflächen-Kugel- oder Kegelform", von E. Meissner, *Physikalische Zeitschrift*, 1913, p. 343.

"Ueber Elastizität und Festigkeit dünner Schalen", von E. Meissner, Vierteljahrschrift der Naturforschenden Gesellschaft in Zürich, 1915, p. 23.

"Festigkeitsberechnung von Ringflächenzchalen", von H. Wissler, Thesis, Zurich, 1916.

"Festigkeitsberechnung von Kugelschalen", von L. Bolle, Thesis, Zurich, 1916; also, abbreviated in *Schweizerische Bauzeitung*, Vol. 66, 1915, pp. 105 and 111.

"Ueber die Festigkeit der Kegelschale", von E. Dubois, Thesis, Zurich, 1917.

"Festigkeitsberechnung von Kugelschalen mit linear veränderlicher Wandstärke", von E. Honegger, Thesis, Lucerne, 1919.

"Ueber eine neue Berechnung der Spannungsverteilung in zylindrischen Behälterwänden", von W. Effenberger, *Beton und Eisen*, 1917.

* "Die Grossmarkthalle in Leipzig, ein neues Kuppelbausystem Zusammengesetzt aus Zeiss-Dywidag-Schalengewölbe," von Franz Dischinger und Hubert Rusch, *Beton und Eisen*, 1929, No. 18, pp. 326-329; No. 19, pp. 341-346; and No. 23, pp. 422-429.

"Kegelförmige Behälterböden,- Dächer und Silotrichter", von F. Kann, Berlin, 1921.

"Neuere Fortschritte der technischen Elastizitätslehre", von L. Föppl, *Zeitschrift für angewandte Mathematik und Mechanik*, 1921, p. 478.

"Drang und Zwang", von A. Föppl und L. Föppl, Munich, 1920, Vol. 2, Chapter 5.

"Formeln zur raschen Berechnung der Biegebeanspruchung in kreisrunden Behältern", von P. Pasternak, *Schweizerische Bauzeitung*, 1925, p. 129.

"Ueber die Festigkeit achsensymmetrischer Schalen", von J. Geckeler, Berlin, 1926; also, abbreviated in *Zeitschrift* des Vereins deutscher Ingenieure, January 30, 1926.

"Zur Festigkeitsberechnung von Hochdruckkesseltrommeln", von E. Meissner, *Schweizerische Bauzeitung*, No. 1, 1926, p. 1.

"Der Spannungszustand gewölbter Böden", von E. Hohn, *Zeitschrift* des Vereins deutscher Ingenieure, Vol. 69, 1925, p. 155.

"Die praktische Berechnung der Biegebeanspruchung in kreisrunden Behältern mit gewölbten Böden und Decken und linear veränderlichen Wandstärken", von P. Pasternak, *Schweizerische Bauzeitung*, Vol. 90, October-November, 1927; also, reprint.

"Kuppeln mit gleichen Normalspannungen", von Th. Pöschl, *Bauingenieur*, 1927, No. 34.

"Beitrag zur Berechnung des zylindrischen Behälters mit veränderlicher Wandstärke", von E. Steuermann, *Beton und Eisen*, No. 15, pp. 286-288; see, also, *Zeitschrift für angewandte Mathematik und Mechanik*, 1925, p. 463.

"Ueber die Festigkeit von Kreiszylinderschalen mit nicht achensymmetrischer Belastung", von Miesel, *Ingenieur Archiv*, Vol. I, No. 1, December, 1929.

DAVID C. COYLE,* M. AM. SOC. C. E. (by letter).†—The tendency of American engineers to insist on rough and "practical" formulas for use in actual work is naturally irritating to many who are accustomed to the everyday employment of the elastic theories, for example, those who have been trained in Continental Europe. There is, however, some little justification for roughly approximate methods, a fact which was brought out in the discussion of the paper by Albert W. Ross, Jr., Jun. Am. Soc. C. E., and Clyde T. Morris, M. Am. Soc. C. E., in 1928, on the modified slope-deflection theory in wind bracing.‡

Briefly, the writer's point of view is one of frank though respectful skepticism as to the relation between the admirable mathematical systems propounded and the data to which they are applied. Concrete on the job is so likely to have occasional lapses from that perfect homogeneity which the elastic theories presuppose. As for the n neutral axes which would exist in a spherical dome anti-symmetrically loaded with a wind of the degree, sin $n\psi$, there is no probable reason to doubt the mathematics of the matter, but it is questionable whether in actual construction a wind of such a special character needs to be considered.

The difference in point of view is very well illustrated by Mr. Floris' comment§ on the trapezoidal distribution of pressures due to unbalanced loads, which he quite correctly states has no logical basis in its analogy to the stresses in a prism. The fact remains, however, that the stresses obviously have a distribution which is of the same general form as the assumed distribution; a reference to his Figs. 10‖ and and 12¶ will indicate at once that the results obtained by advanced statics are indistinguishable at small scale from those shown by the rough formulas. It is evident that the difference between the trapezoidal distribution and any other probable distribution of the stresses is similar to that between a five centered oval and an ellipse. The mathematics

* Designing Engr., Gunwald Aus Co., New York, N. Y.

† Received by the Secretary, January 25, 1930.

‡ *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1395.

§ *Loc. cit.*, April, 1930, Papers and Discussions, p. 809.

‖ *Loc. cit.*, p. 811.

¶ *Loc. cit.*, p. 813.

of the two are very unlike, but the numerical results come very close to each other. As for the wind effects shown in Fig. 11,* it is permissible to wonder whether the purity of the mathematics may not have obscured the inconsequence of the practical application.

The older theories of domes on the Continent seem to have involved originally the assumption of bending moments in the ribs of a pyramoid. No doubt the development of advanced statics which has justified mathematically the use of a center-line stress system has seemed to be a radical liberation. It may be fair to remark, however, that the common theory in the United States has always been too inexact to take the secondary bending into account. For this reason Mr. E. Schmitt's formulas for pyramoids, published in 1904† were based on the absence of bending in the ribs. The further elimination of bending from the shell between the ribs or "hips" is interesting, but the results are perhaps unfortunate. There is a constant danger that the growth of mathematical methods, the accuracy of which exceeds the homogeneity of the materials, may lead to economy which is based on an illusion. The domes of the great market in Leipzig, Germany, are illustrative of this point. By a mathematical process the builders have convinced themselves that practically no thickness at all was required. Similarly, it has always been recognized that the theoretical required thickness of a spherical dome is practically zero, the live load being small. It has not followed, however, that zero thickness was the best for the purpose.

In the building of actual structures, the proper function of mathematics is to check the general validity of the practical formulas used for the work itself; but if the accuracy of the mathematical methods used is allowed to become the main object, it is dangerous. The stresses having been computed to four places of decimals, what if a storm blows the covers off some night and the previous day's pour is frozen? Or what if there is sugar in one load of sand? Or what if two out of sixteen footings settle $\frac{1}{8}$ in. more than the other fourteen? Or how about the indications that in some concrete the ratio between concrete and steel stresses is not equal to n, but to a function of time? What happens to the elastic theory if n is incorrect?

It is evident that the formulas given in this paper have a real approximate relation to the actual stresses in the structure. The close correspondence of the general shape of the stress diagrams with those of the more accurate theories indicates that the paper is not widely at variance with the facts. If their algebraic derivation is free from error, these formulas are suitable to give the engineer an idea of whether his design is well on the safe side. At the same time, they are free from the disadvantages of the high mathematics, of taking an unconscionable time, and of producing the illusion that their results are more accurate than their data.

* *Proceedings*, Am. Soc. C. E., April, 1930, Papers and Discussions, p. 812.
† *Transactions*, Am. Soc. C. E., Vol. LII (1904), p. 293.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

REGULATION OF LEVELS, FLOW, AND NAVIGATION, NIAGARA RIVER:

SUMMARY AND CONCLUSIONS OF VARIOUS STUDIES

Discussion*

By F. G. Ray, M. Am. Soc. C. E.

F. G. Ray,† M. Am. Soc. C. E. (by letter)‡—The regulation of the Great Lakes has been a subject of discussion for many years. At first glance it appears to be a simple engineering proposition. There are, however, opposing interests to be served and restricting conditions to be met which offer many complications. The author has clearly set forth the fundamental considerations in reaching a solution and his conclusions show what little betterment may be expected in attempting to improve upon the regulation that Nature has provided. Regulation is feasible as he has shown, but it must be kept within a narrow range to avoid unreasonable damage to those whose interests are diametrically opposed to the interests benefited. The net result is small as measured by the cost. The time may come when regulation is warranted, but in the writer's opinion that time has not yet arrived.

The subject is highly interesting and should commend itself to the study of hydraulic engineers. If regulation is ever to be adopted, every practicable plan should be conceived and considered in advance. Regulation of Lake Superior is now in successful operation. The program to be followed and the benefits to be derived with regulation of Lakes Erie and Ontario are a matter for study, but the design and construction of regulating works for the Niagara and St. Lawrence Rivers offer no serious difficulties. Regulation of Lakes Michigan and Huron, however, is a more intricate problem. These lakes discharge through 90 miles of channel which carries a flow of 200 000 cu. ft. per sec., with an average slope of less than 0.1 ft. per mile. There is an immense traffic through this channel which must not be seriously hampered

* Discussion of the paper by George B. Pillsbury, M. Am. Soc. C. E., continued from January, 1930, *Proceedings*.

† Prin. Engr., U. S. Lake Survey, Detroit, Mich.

‡ Received by the Secretary, January 30, 1930.

by obstructions or largely increased velocities. These are conditions that must be met in regulating the two lakes in question.

In analyzing the practicability of regulating schemes it has been customary to test them by use of supply factors and lake levels of record since 1860. The plans discussed by Colonel Pillsbury were so tested by him up to 1925. The writer has extended the so-called partial regulation scheme for Lakes Michigan, Huron, and Erie through to the end of 1929. It is interesting to note that the high-water peak which resulted in considerable damage on Lakes Michigan and Huron in 1929 would have been exceeded by 1 ft. and Lake Erie would have gone 3 in. above the highest previous level since 1860, if this particular plan had been in operation. Regulation projects based on supply factors now available are accordingly further restricted or subject to less favorable results.

The author has shown the effect of Lake Superior regulation on the levels of the several lakes from 1914 to 1925, inclusive. This study has also been extended to include 1929. It has been found that the maximum stage on Lake Superior in 1928 was about 7 in. below what the unregulated level would have been; also that the effect of the regulation on the maximum stages of Lakes Michigan, Huron, and Erie in 1929 was to raise each of these lakes slightly less than 2 in.

In proposals for compensating works in the outlets of the several lakes, it is usually considered that such works should be so designed as to compensate for all losses in lake level due to diversions and to changes in original capacity of outlet. The amount that Lake Erie has been lowered and the corresponding amount of compensation required in the Niagara River are known with little uncertainty. In the case of Lakes Michigan and Huron the effect of diversions are determined by means of measured increments of flow in the outlet. The possibility of changes in capacity of outlet and the lowering effect on Lake Huron of channel enlargements in the St. Clair and Detroit Rivers, prior to the beginning of discharge measurements in 1899, cannot be determined by direct observation, except through comparison of past and present relation of levels on Lake Huron and Lake Erie. The period of relatively low water from 1890 to 1926 left considerable doubt as to the showing of such comparison. The high stages of 1929 have done much to strengthen the evidence.

Fig. 14 shows a comparison of lake levels by plotting 5-year means with the levels of Lakes Michigan and Huron as ordinates and the levels of Lake Erie as abscissas. The observed levels have been corrected for earth tilt to corresponding levels at the outlets. It will be noted that 5-year means from 1870 to 1919 show little departure from a straight-line relationship, except for the two groups, 1890-1894 and 1910-1914. The local supply to Lake Erie was notably excessive in 1913. Lake Erie levels in 1890 appear relatively high. The reason has not been investigated. When the two years, 1890 and 1913, are omitted, the levels for the remaining 48 years of the 50-year period are remarkably consistent, and show no evidence of change in relative height, the levels for 1925-1929, however, fall about 5 in. below the 1870-1919 line

and the single years, 1926, 1927, and 1929, are about 6½ in. below this line. The 1929 levels are well up among those of early years. The evidence indicates that Lake Huron held its proper level until after 1919 and has been lowered since that time by about 6 in. This amount is subject, of course, to some error resulting from inequalities in local supply.

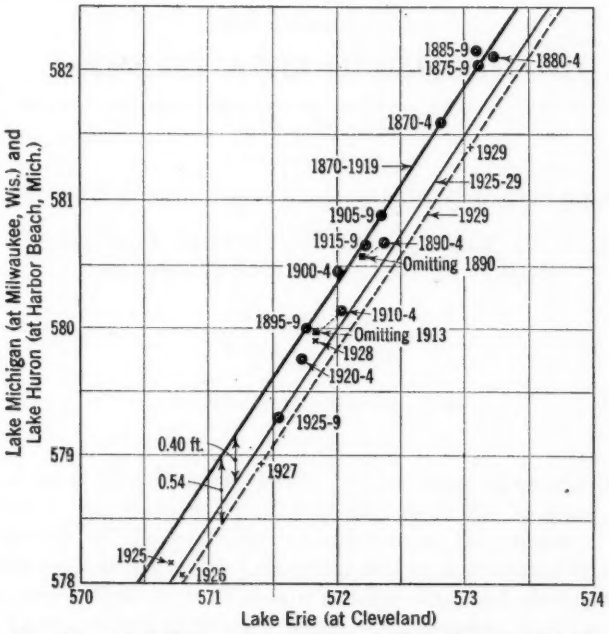


FIG. 14.—COMPARISON OF LEVELS IN LAKES MICHIGAN-HURON AND LAKE ERIE.

Soundings in the head of St. Clair River in 1925 showed a marked enlargement in cross-section due to gravel dredging in the vicinity of Point Edward and below. The amount of enlargement indicates a resultant lowering of about 4 in. on Lake Huron. Discharge measurements and analysis of slopes show the effect to be about 3½ in. The amount of lowering not accounted for by this enlargement is apparently not more than about 3 in. A continuation of high water on the lakes will afford safer and surer conclusions. However, in the writer's opinion, the effect on Lake Huron levels of channel improvement and other changes in the St. Clair and Detroit Rivers is now known quite definitely.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

FRICTION OF SEWAGE SLUDGE IN PIPES

PROGRESS REPORT OF COMMITTEE OF THE SANITARY ENGINEERING DIVISION

Discussion*

By William Clifford, Esq.

William Clifford,† Esq. (by letter).‡—The progress report§ is a welcome contribution to the study of friction of sewage sludge in pipes and a most useful summary of the available information on the subject. The experimental work undertaken by the Committee will be appreciated by all engineers who know how difficult it is to obtain reliable data, and although the results appear to be painfully negative the writer would suggest that the members of the Committee have "builded better than they knew".

Regarding English experience,§ it should be stated that the conveyance of sewage sludge through iron pipes for short distances became necessary with the adoption of sludge presses in the early Nineties, and when, in 1897, the City of Manchester, England, decided to barge the sludge to the sea the pipe line from the sewage works to the Ship Canal was laid. It was a 10-in. pipe, ¾ mile long. The report states‖ that "Clifford * * * does not give either moisture or temperature". In reply, the writer would refer the Committee to Table V and elsewhere in the paper mentioned.¶ The water content is there given as 90 per cent. It would be interesting to know what "physical characteristics"** are destroyed by passing sludge through a meat chopper. The writer feels that the Committee examined and rejected,†† on insufficient

* Discussion on the Progress Report of the Committee of the Sanitary Engineering Division on Friction of Sewage Sludge in Pipes, continued from November, 1929, *Proceedings*.

† Sewage Works Mgr., Wolverhampton Corporation, Wolverhampton, England.

‡ Received by the Secretary, December 11, 1929.

§ *Proceedings*, Am. Soc. C. E., September, 1929, Papers and Discussions, p. 1773.

‖ *Loc. cit.*, p. 1783.

¶ "Friction of Sewage Sludge in Pipes," by William Clifford, *Transactions*, International Conference on San. Eng., 1924.

** *Proceedings*, Am. Soc. C. E., September, 1929, Papers and Discussions, p. 1784.

†† *Loc. cit.*, p. 1774.

grounds, the suggestion he presented some years ago that the general law of the flow of fluids could be applied to give a first approximation to the loss of head (pressure drop) occasioned by the flow of sewage sludge in pipes. On the other hand, the efforts of the Committee to find a suitable formula or a suitable constant in an empirical formula applied to water, have been fruitless. In these circumstances it may be worth while to examine the evidence for viscous flow more closely.

The relation between the several material factors affecting the flow of fluids in pipes is known, and it is the value of only one factor, kinematic viscosity, that remains to be determined; this for homogeneous fluids is measured by the absolute viscosity divided by the specific gravity. In the case of sewage sludge, the determination of the absolute viscosity by direct means is out of the question and the problem is how to measure the kinematic viscosity by indirect means. Two methods are available: (*a*) By comparing the flow of sludge with the flow of a homogeneous fluid of known viscosity; and (*b*) by careful determination of the friction losses in a pipe line; that is, the measurement of the hydraulic slope (hydraulic gradient or pressure drop) and using the formula for the calculation of the kinematic viscosity.

The difficulty about the variable character of sludge is more apparent than real. After all, the engineer is concerned with the maximum viscous condition of sludge to be provided for. The disposal of sludge with a water content of 90%, appears to be quite a general occurrence in England. The kinematic viscosity which the writer obtained for 90% sludge was determined by direct comparison of the time of efflux of sludge from a hopper-shaped vessel with that of a homogeneous liquid (glycerine and water) under like conditions (Method (*a*)).

The value obtained for the kinematic viscosity of sludge with a water content of 90%, was 0.001055, in foot-pound-second units. As a check on this the data for a 4-in. pipe line used for conveying sewage sludge, Method (*b*) was available. The values as to water content, velocity in pipe line, and pressure drop were averages; nevertheless, accepting these values and working backward, a kinematic viscosity of the order of 0.001 sec-ft. was obtained. In these circumstances and until further information is available the value of 0.001055 sec-ft. is offered as a tentative value of the kinematic viscosity of 90% sludge, to be varied as more accurate information is obtained.

Apart from the existing pipe lines in England the progress report gives* data relating to tests, in Toronto, Ont., Canada, Syracuse, N. Y., Baltimore, Md., and Chicago, Ill.

The Toronto tests are ruled out as unreliable and not consistent.† The results of the Baltimore† tests are not recommended, and the Syracuse tests† (1926) deal with 99.5% watery sludge and are not useful in the present connection. This leaves the tests at Chicago on the 8-in. and the 12-in. pipes for consideration.‡

* *Proceedings*, Am. Soc. C. E., September, 1929, Papers and Discussions, p. 1780, *et seq.*
† *Loc. cit.*, p. 1775.
‡ *Loc. cit.*, p. 1785.

The low results observed with the 12-in. pipes call for further investigation. On the whole the results for the 8-in. pipe appear to be accepted as the more reliable. The writer, therefore, has taken the value of the loss of head on the 8-in. pipe line from Fig. 9* for 90% moisture at four different velocities. These are shown in Table 5, Column (2), and in Column (3) are the corresponding figures obtained on the assumption that the kinematic viscosity of the sludge was 0.001055. The results are much closer than could have been anticipated.

TABLE 5.—COMPARISON OF HEAD LOST IN 8-INCH PIPE LINE AT CHICAGO, ILL. (Moisture, 90%.)

Velocity, in feet per second. (1)	Loss of Head, in Feet per 100 Feet.	
	From Fig. 9. (2)	Calculated. (3)
1.0	0.30	0.23
2.0	0.47	0.47
3.0	0.75	0.75
4.0	1.20	1.60

As there are no data available for the kinematic viscosity of sludge with 88% moisture, it is reasonable to assume that it would be greater than 0.001 and as a trial the writer assumed that it would be about 0.0015 sec-ft. Using this value the results given in Table 6 were obtained.

TABLE 6.—COMPARISON OF HEAD LOST IN 8-INCH PIPE LINE AT CHICAGO, ILL. (Moisture, 88%.)

Velocity, in feet per second. (1)	Loss of Head, in Feet per 100 Feet.	
	Actual from Fig. 9. (2)	Calculated. (3)
1.0	0.57	0.33
2.0	0.70	0.67
3.0	0.95	1.00
4.0	1.30	1.37

Although the calculated results are low at average low velocities (about 1 ft. per sec.) and high at velocities faster than 3 ft. per sec., nevertheless, as a first approximation and until better data are available the writer suggests that for sludge with 90% moisture a kinematic viscosity of 0.001055 sec-ft. may be used as a means of computing the pressure drop or hydraulic slope.

He is fully aware of the difficulties of determining the coefficient of friction and appreciates the excellent work done at the Calumet Works in Chicago

* *Proceedings*, Am. Soc. C. E., September, 1929, Papers and Discussions, p. 1788.

to elucidate this point; but a difficulty in determining a factor directly does not seriously affect the question when an indirect method of measurement is available.

In the light of the relationships described herein the writer would suggest that further attempts be made to explore the various ways of measuring the physical qualities of sludges and to correlate these with the friction losses obtained in practice from different sizes of pipes.

The law of fluid resistance may be stated as:

$$R = \rho\, V^2\, f\left(\frac{V\,d}{\nu}\right)$$

or,

$$\frac{R}{\rho\, V^2} = f\left(\frac{V\,d}{\nu}\right) \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

in which:

R = resistance per unit area.
d = diameter, in feet.
ρ = density.
V = velocity, in feet per second.
ν = kinematic viscosity = $\dfrac{\text{absolute viscosity}}{\text{density}}$.
g = gravity constant = 32.2 ft. per sec. per sec.
m = hydraulic radius = $\dfrac{d}{4}$.
i = hydraulic slope.

The values of $\dfrac{R}{\rho\, V^2}$ for various fluids at various velocities have been determined experimentally by Stanton* and Pannall and others, and have been plotted against the corresponding values of $\dfrac{V\,d}{\nu}$. The writer has used these values and extended them to the lower values of $\left(\dfrac{V\,d}{\nu}\right)$ by plotting on logarithmic paper as suggested by Davies and White.†

The data for the evaluation of the hydraulic slope are given in Table 7.

The values of the Reynolds criterion, $\dfrac{V\,d}{\nu}$, indicate that a fluid (90% sewage sludge) having a kinematic viscosity of 0.001055 flows in an 8-in. pipe with a laminar flow up to about 3 ft. per sec. At 4 ft. per sec., the flow is probably turbulent, but it would be desirable to ascertain this by experiment. With the more viscous fluid (88% sludge) a laminar flow is indicated up to at least 4 ft. per sec.

On the other hand thin sludges with a kinematic viscosity differing very little from water (0.00001 sec-ft.) will flow with turbulence at even 0.1 ft.

* "Friction," by T. E. Stanton, 1923, Fig. 12.

† "A Review of Flow in Pipes and Channels," by S. G. Davies and C. M. White, *Engineering*, Lond., July 20, 1929, p. 70.

per sec. $\left(\frac{V\,d}{\nu} = 6\ 667\right)$. The foregoing consideration calls for further information as to:

(1) The values of the kinematic viscosity to be used for various sludges;
(2) Determination of the conditions of flow (laminar or turbulent) in pipes of various sizes and at radius velocities; and,
(3) Observation and collation of records of pressure drop in actual pipe lines.

TABLE 7.—EVALUATION OF HYDRAULIC SLOPE.

V, velocity, in feet per second.	d, diameter, in feet.	$\left(\frac{V\,d}{\nu}\right)$	$\frac{R}{\rho\ V2}$	$\frac{\frac{R}{\rho\ V2}}{32.2} = \frac{m\,i}{V2}$, in second-feet units.	$i = \frac{V2}{m} \times \frac{\frac{R}{\rho\ V2}}{32.2} = V2 \times 6 \times (f)$
KINEMATIC VISCOSITY (ν), 0.001055 SECOND-FEET					
1	2/3	632	0.0125	0.00039	0.0023
2	2/3	1 264	0.006	0.00019	0.0047
3	2/3	1 896	0.0045	0.00014	0.0075
4	2/3	2 528	0.0054	0.00017	0.0165
KINEMATIC VISCOSITY (ν), 0.0015 SECOND-FEET.					
1	2/3	444	0.018	0.00055	0.0033
2	2/3	888	0.008	0.00027	0.0067
3	2/3	1 333	0.006	0.00018	0.0118
4	2/3	1 776	0.0045	0.00014	0.0137

Possibly works in which oil, glycerine, or similar viscous fluids of known density and viscosity are being pumped, could supply data that would be helpful in this connection.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

ESSENTIAL FACTS CONCERNING THE FAILURE OF THE ST. FRANCIS DAM

REPORT OF COMMITTEE OF BOARD OF DIRECTION

Discussion*

BY MESSRS. B. F. JAKOBSEN, M. H. GERRY, JR., F. B. MARSH, AND F. C. FINKLE.

B. F. JAKOBSEN,† M. AM. SOC. C. E. (by letter).‡—The spillway crest of the St. Francis Dam was at Elevation 1 835 and the base at Elevation 1 625 (see Fig. 2§), where the dam was 176 ft. thick. The parapet was about 3 ft. higher, so that the height of the dam was 213 ft., and not 205 ft., as stated.‖

In Fig. 3,¶ the section, which remained standing, is that marked (1) in the elevation and not the one indicated in the diagram.

The writer had visited the dam two or three times during construction and visited the site a few days after the disaster. He had noted the questionable character of the foundation and the haphazard manner of making concrete, but he had been unable to secure a drawing of the dam until after it had failed.

A gravity dam may be considered with regard to the following headings: (1) Foundation; (2) grouting of foundation; (3) cut-off wall; (4) drainage of foundation and dam; (5) contraction joints; (6) quality of concrete; (7) uplift allowance; and (8) design, that is, sufficiency of dimensions, weight, etc. The St. Francis Dam was admittedly defective with respect to the first five items. If proper drainage had been provided, excessive leakage would have given warning long before the reservoir was full and long before any danger was imminent, as all inflow into the reservoir could have been stopped. It is worth noting, for future reference, that an analysis of the water which

* Discussion of the Report of the Committee of the Board of Direction on Essential Facts Concerning the Failure of the St. Francis Dam, continued from January, 1930, *Proceedings*.

† Cons. Engr. (La Rue & Jakobsen), Los Angeles, Calif.

‡ Received by the Secretary, November 6, 1929.

§ *Proceedings*, Am. Soc. C. E., October, 1929, Papers and Discussions, p. 2150.

‖ *Loc. cit.*, p. 2148.

¶ *Loc. cit.*, p. 2152.

leaked past the dam showed that it contained many times as much dissolved solids as the water which was being diverted into the reservoir.

Regarding the concrete, three samples tested by the Governor's Committee showed a strength of from 2 007 to 2 717, with an average of 2 429 lb. per sq. in. The concrete was two years old* and 1.12 bbl. of cement had been used per cubic yard of concrete.† It has been reported that more than three samples were secured by the Governor's Committee and that all but three tested very low and were not included in the official report. Information on this point would be of considerable interest. The concrete made at the Pacoima Dam, while the writer was in charge, averaged 2 650 lb. per sq. in. at 28 days‡ and at the age of 2 years this had increased to 5 300 lb. per sq. in. About 1 bbl. of cement was used per cubic yard of concrete and the water-cement ratio was carefully controlled. The concrete at the St. Francis Dam was made from pit-run river-bed gravel and its strength of 2 429 lb. per sq. in. at 2 years shows that it was distinctly inferior. It is quite possible that a good quality of concrete might have held the dam together, so that a foundation blow-out might have given warning of the impending disaster. This is a possibility because the dam had been constructed without contraction joints and had probably not cooled sufficiently to produce shrinkage cracks clear through the structure.

The dam was designed without any allowance for uplift, nor was there any allowance made for the water rising above the spillway lip. There is difference of opinion as to whether or not uplift should be allowed for in the design of a gravity dam, when every possible precaution has been taken to minimize it, and the dam is founded on good rock. The writer believes that considerable uplift should always be provided for in gravity dams, because in spite of every precaution, there is no doubt always some uplift present;§ because partial stoppage of the drainage passages is likely to occur;‖ because generally no account has been taken of earthquake shocks, which may materially increase the load on a dam;¶ and, finally, because the factor of safety of all but the most conservative gravity dam designs is alarmingly small.**

Leading engineers have emphasized the danger of neglecting uplift.†† The writer has shown elsewhere that the worst condition exists when full uplift acts over the up-stream two-thirds of the base.‡‡ This is, of course, a limiting case which cannot actually occur. The writer made a calculation for the Mulholland Dam, which is identical with the St. Francis Dam, except for a small difference in the radius, and found§§ for a section, 135 ft. high, with curvature taken into account,‖‖ with water level with the top of the dam, and

* Report of the Commission Appointed by Governor C. C. Young, dated March 24, 1928.
† *Proceedings*, Am. Soc. C. E., October, 1929, Papers and Discussions, p. 2148.
‡ *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 585.
§ *Loc. cit.*, Vol. 93 (1929), pp. 1527, 1559, 1563, and 1578.
‖ *Loc. cit.*, Vol. 91 (1927), p. 249.
¶ *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 220.
** *Transactions*, Am. Soc. C. E., Vol. 91 (1927), p. 268.
†† *Loc. cit.*, p. 265; *Loc cit.*, Vol. LXXV (1912), pp. 148, 154, 212, and 218.
‡‡ *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 224.
§§ *Loc cit.*, p. 225.
‖‖ "Notes on Arched Gravity Dams", *Transactions*, Am. Soc. C. E., Vol. 92 (1928), p. 766.

with no uplift allowance, that the factor of safety against overturning was only 1.04. This is entirely insufficient. The small section is also at a disadvantage, when sliding is considered.

These many deficiencies notwithstanding, the Committee appointed by the City Council states:* "The type and dimension of the dam were amply sufficient if based on suitable foundation."

This Committee believes then that a design, which has a factor of safety of 1.04, is not merely sufficient, but amply sufficient, although this factor includes no allowance for uplift and there was no serious attempt to minimize this by grouting, cut-off wall, and drainage. The Committee apparently has faith in the absolute accuracy of the assumptions involved in gravity dam design, since a margin of only 4% is characterized as "amply sufficient".

The Governor's Committee states:†

"There is nothing in the failure of the St. Francis Dam to indicate that the accepted theory of gravity dam design is in error or that there is any question about the safety of concrete dams designed in accordance with that theory when built upon even ordinarily sound bed-rock. On the contrary, the action of the middle section which remains standing, even under such adverse conditions, is most convincing evidence of the stability of such structures when built upon firm and durable bed-rock."

The first question which occurs to an engineer visiting the site of the failure is, "why did a part of the dam remain standing, while the remainder failed?" The writer has investigated this question carefully, because this is the crux of the whole matter, and the only plausible explanation completely refutes the assertions made by this Committee. In order to understand this question properly, it is necessary to bear in mind, that the reservoir had been full for a week before the failure and had been nearly full for many months, as stated:‡

"Water was stored to Elevation 1 832, or within 3 ft. of the spillway level, May 10, 1927, and during the remainder of that year at no time fell below Elevation 1 813, * * *. Beginning with November 1, 1927, the level was gradually raised, and by March 5, 1928, a week before the catastrophe, reached Elevation 1 835, full reservoir capacity * * *."

If the design of the dam had been such that each section had the same stability, then the only reasonable explanation would be that the foundation was better under the high section; but the higher sections had a much greater stability, as Table 1 shows.

The stability of a dam, such as the St. Francis, is roughly proportional to the ratio of base width to height. At a distance of 138 ft. from the crest, this ratio is 0.668 and when no uplift is allowed for, this section has a factor of safety against overturning of about 1.04, which means that the resultant lies practically in the down-stream middle-third point. No part of the St. Francis Dam, which remained standing, has a ratio of less than 0.763.

The maximum section with a base width of 176 ft., has a factor of safety against overturning of about 1.4 when no allowance is made for uplift. This

* *Proceedings*, Am. Soc. C. E., October, 1929, Papers and Discussions, p. 2159.
† *Loc. cit.*, p. 2155.
‡ *Loc. cit.*, p. 2150.

is a margin of 40%, or ten times as much as for the section 135 ft. high. With full uplift under the up-stream 68 ft. of the base, this maximum section would still be barely stable; that is, the resultant would lie in the down-stream middle-third point. The difference in the factors of safety for the various sections, ranging from 1.04 to 1.4, the writer believes, fully explains why part of the dam remained standing. Furthermore, the fact that only a part of this maximum section remained, emphasizes the opinion that a factor of safety of 1.4 is, to say the least, by no means excessive.

TABLE 1.—DIMENSIONS OF THE ST. FRANCIS DAM.

Distance from crest, in feet.	Base width, in feet.	Ratio of base to height.
68	42.54	0.627
103	67.11	0.650
138	92.13	0.668
163	114.25	0.701
193	147.10	0.763
213	176.00	0.828

The writer's explanation of the manner of failure is, that uplift tilted the dam down stream, the sections of less height leaning for support on the sections of maximum height, which had a much greater stability. As uplift increased, two vertical radial planes of fracture were produced, and the section of dam between them slid out. The local character and condition of the foundation and planes of weakness due to shrinkage cracks and possibly tension, extending partly or completely through the dam, determined the location of the planes of fracture. Any one witnessing the failure would probably have seen only a sliding of the dam on its foundation and a simultaneous crumbling of the foundation, the tilting or rotation being difficult to discover unless careful measurements were made. Which wing went out first would seem to be immaterial. Moreover, the knowledge of the condition of the foundation, shrinkage cracks, and tension in the dam, etc., at the instant of failure are entirely insufficient to form the basis for sound conclusions.

The Committee appointed by the Governor of California refers to "the accepted theory of gravity dam design".* By this is probably meant an arched section without any allowance for uplift, and computed on the assumption that it is straight instead of curved. Analysis of one gravity dam of this shape, assuming no uplift or arch action, disclosed a tension of 40 lb. per sq. in. at the heel when the reservoir is full.†

The "accepted theory" also assumes that the vertical stresses on horizontal sections are linear. It is the writer's opinion that as regards uplift, the majority of competent engineers do not agree with this Committee. A great number of quotations and references could be brought forward to substantiate that assertion. The same is true, although possibly to a lesser extent, of the other two assumptions.

* *Proceedings*, Am. Soc. C. E., October, 1929, Papers and Discussions, p. 2155.
† *Transactions*, Am. Soc. C. E., Vol. 92 (1928), pp. 72 and 835.

The writer has already discussed the fundamental weakness of depending on arch action* and here would simply point out that arch action did not save the thin gravity sections of the St. Francis Dam, and furthermore, for the part which remained standing, arch action can claim no credit. After a careful investigation the writer has come to the conclusion that if the entire St. Francis Dam had been designed with a base ratio of about 80%, although this would not make a safe dam, there would have been no sudden failure. The facts that the reservoir had been nearly full for many months and that part of the middle section remained standing, support such a conclusion. Furthermore, the water pressure on the foundation, the horizontal thrust of the water, and the overturning moment of the water vary as the first, second, and third power of the depth of water, respectively.

The difference in yardage between a section, which is barely stable when there is no uplift, or, say, about 67% base width for 140 concrete,† and one which can withstand full uplift, or, say, about 90% base width, is only about 34 per cent. The resulting difference in cost is about 25%, and this is by no means an unsurmountable obstacle, in view of the fact that the smaller section has no positive margin of safety.

The theory of gravity dam design also involves the assumption that the vertical stresses on horizontal sections are linear; it is by means of this fundamental assumption that stresses are determined. As far as the writer knows, there has never been presented either theoretical or experimental evidence of the correctness of this assumption. It was borrowed from the theory of flexure of beams and it has been shown by experience that it is true for long beams; but a gravity dam is not a long beam, since its thickness is never less than about 67% of its length and, moreover, it is not a uniform section. Some years ago the writer stated:‡

"Until the actual stresses in high gravity dams have been determined from experiments and theory, engineers will do well to bear in mind the possible influence of the inaccuracies in the fundamental assumptions."

Again, it has been stated:§

"Modern dam design * * * recognizes the fact that Rankine's criterion, namely, that the resultant of the forces must fall within the middle third of the section, is not sufficient to prevent the occurrence of tension in the mass of the dam."

Recently, the writer has investigated the stress distribution in gravity dams and found that it does not agree with the usual assumption of linear stress distribution. Tests, as far as they go, appear to confirm the writer's theoretical investigations.

The St. Francis Dam deviated in several important respects from ordinary practice, notably in neglecting uplift, while taking no precaution to minimize it. The neglect of uplift led to a design, which had virtually no margin of

* *Transactions*, Am. Soc. C. E., Vol. 92 (1928), pp. 772 and 835.

† In the report of Governor Young's Committee, referred to previously, the weight of three samples of concrete is given as 137.4, 142.8, and 141.7, or an average of 140.6 lb. per cu. ft. These samples were wet from the coring process.

‡ *Transactions*, Am. Soc. C. E., Vol. 91 (1927), p. 254.

§ *Loc. cit.*, p. 255.

safety, and the defective foundation developed considerable uplift and destroyed the dam. It is true that there are a number of gravity dams standing, which have sections no larger than the upper part of the St. Francis Dam, but the mere fact that a dam has not failed, does not prove it safe. Such a dam may never have been subjected to full load for a considerable period of time; or, an unusual occurrence, such as a flood or an earthquake shock, which would produce a failure,* may not have happened while the reservoir was full. The Habra Dam, which has failed three times,† stood for nearly fifty years, from 1881 to November, 1927, when it failed due to a heavy flood, which brought the water level about 12 ft. above the spillway crest. The St. Francis Reservoir was full for one week before the disaster occurred and had been nearly full for many months.‡ If, a day or two before the dam failed, the reservoir level had been drawn down about 10 to 15 ft., this dam would most likely be standing yet, testifying to its proven safety.

Recommendations.—This failure illustrates the danger of designing on the "ragged edge", so that no margin of safety is provided for an unusual condition, which in this case was the defective foundation. In order to avoid a recurrence of such a disaster the Society should appoint a committee of competent engineers to formulate rules for minimum safety of design, and should attempt to establish an accepted theory, which does not exist at present. Such a code would also be invaluable to the various State officials, who have supervision over dams.

Conclusions.—Based on the foregoing remarks, the writer offers the following conclusions:

1.—The St. Francis Dam failed because it was not properly designed, its dimensions being wholly insufficient to withstand the horizontal water pressure plus a small amount of uplift.

2.—Due to the poor rock, the foundation became saturated at an early date, producing sufficient uplift to destroy the dam. If the dam had been on sound and durable bed-rock, but lacking drainage and grouting of the foundation, as did the St. Francis Dam, the destruction would have been delayed, because it would have required more time to produce sufficient uplift.

3.—If the dam had been designed for full uplift, the additional cost being about 25%, there would have been no sudden failure, and probably no loss of life.

4.—If the foundation had been provided with drains, as is customary, excessive leakage would have given ample warning long before there was any possibility of danger, and loss of life would have been avoided.

M. H. Gerry, Jr.,§ M. Am. Soc. C. E. (by letter).‖—At this late day (February, 1930) a discussion in this forum of the St. Francis disaster can be justified only by the expectation that out of the lessons it teaches will come a logical method of designing gravity dams with greater assurance of adequate safety.

* *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 219.
† *Loc. cit.*, p. 227.
‡ *Loc. cit.*, October, 1929, Papers and Discussions, p. 2150.
§ Cons. Engr., San Francisco, Calif.
‖ Received by the Secretary, December 4, 1929.

Such a debacle as this, with its attendant train of death, vast property loss, and the destruction of public confidence in the Engineering Profession, should not be passed by lightly with the excuse that it was an event resulting from a mere oversight of natural conditions. To any one giving the matter serious thought it must be apparent that there are more adequate reasons for this failure.

One plain truth is that the Engineering Profession at large has not realized how narrow is the margin of safety in gravity dams as at present designed; nor has the general public any conception of the extent of the hazard involved. It would seem, therefore, to be the professional duty of engineers with experience in this line, to discuss the subject constructively.

Pursuant to this thought, after carefully weighing all known facts appertaining to the catastrophe, and having examined the site and considered the opinions of many engineers and geologists, the writer has arrived at the following conclusions:

(1) The destruction of the St. Francis Dam was caused by the sliding of the end sections, due solely to the fact that the water pressure at full head, produced forces far greater than the frictional resistances holding these parts in position. The fundamental reason for the failure was defective design.

(2) It was not a foundation failure in any sense whatever, evidence being entirely lacking of any settlement, or erosion of the rock, until after the dam had slipped from its base.

(3) The structure as built was of inadequate dimensions and improper form and did not provide a reasonable margin of safety, regardless of the character of the rock on which it stood.

(4) The major hazard in this dam arose from a tendency to slide on its base, resulting from present methods of design, which are especially deficient in this respect.

(5) The practice now in vogue for designing high masonry dams of the gravity type is obviously defective and incomplete, and should be replaced by more scientific methods, embracing all the factors entering the problem.

In order to substantiate these five conclusions it is proposed now to examine the facts in evidence.

The Foundations.—This structure was reared on the surface of the country rock, its base conforming closely with the natural section of the canyon. Some excavation was made, but this consisted mainly in removing the talus and weathered surface rock, by sluicing and steam-shovel methods. Over the entire floor of the gulch the dam stood on a nearly level base, but at either end it clung to hillside slopes of from 30 to 50 degrees. The footings of the east wing coincided closely with the 45° angle of the slippery cleavage planes of the mica schist, while at the westerly end of the dam, the base rested on a steeply inclined plane of conglomerate, which presented a slimy surface when wet.

The schist at the site is a rock of fair quality for foundation purposes. It has good strength in compression and is permanent in character. Its chief weakness is in shear on planes of schistosity, and here, also, its coefficient of friction is very low (probably less than 0.5). Although traversed by many small

joint planes near the surface, this rock was found to be remarkably tight against water leakage. Like all other rocks, it permitted some seepage, but required considerable time for complete saturation.

The brown conglomerate sandstone, which supported in part the westerly wing, is of less strength and permanency than the schist. It weathers easily on the surface, but when confined at some little depth, apparently it has ample strength to support such loads as were placed upon it. This rock developed small leakage but a little more seepage than the schist.

Notwithstanding the undesirable characteristics of these rocks there was no settlement of any part of the dam, nor was there any scouring beneath the base, prior to the actual collapse of the wings; therefore, it must be admitted that the foundation materials did not yield and were not the cause of this failure.

Some Facts About the Structure.—The dimensions and placement of the dam are shown quite fully in the report of the Committee and in other published reports.

In plan it was curved on a radius of 500 ft. and it was thought, apparently, that this form provided additional strength. To the contrary, it weakened the structure by reducing the weight of radial sections and it resulted also in a base under certain parts of the wings, having a decided slope down stream. This last is always a dangerous condition for a gravity dam.

From slightly above Elevation 1 700 the width of the dam at the base was less than two-thirds of the height. Below this elevation the batters increased to such an extent that the maximum section in the bottom of the gulch had a base width of 82% of the height. However, it should be noted here that the weight of the maximum section was only equivalent to a normal triangular section having a base width of 70% of the height.

This structure was admittedly designed on the common theory that a dam is safe if the resultants of all the forces assumed to be acting fall within the middle thirds of all horizontal sections, including the base. With this went the further assumptions of no uplift and a permissible sliding factor of 0.75.

It was unquestionably the honest opinion of the designers that this dam was entirely safe and they would have said (as has been repeatedly said of other dams of no better section) that it had a "factor of safety of more than two."

Notwithstanding this belief, the evidence shows that it had no margin of safety at all and in certain parts was in unstable equilibrium even before full head was applied. Such a situation can be reasonably explained only by saying that it arose out of professional reliance on a theory and practice either wrong in principle or not sufficiently broad to cover all cases.

Here it should be added that there are no high gravity dams of masonry in existence that have a "factor of safety of more than two"; if by such statement is meant that they would not fail by sliding (if for no other reason) until more than a double amount of fluid pressure had been applied.

Conditions Preceding the Failure.—No one now living saw this dam go out. It burst with great suddenness and without immediate warning. The last person who observed it in place is reported to have passed along the adja-

cent road about ½ hour before the failure and to have noted a watchman standing on the top; but nothing of unusual or alarming nature was then apparent. Later than this, there is no personal record of what happened, for the watchman and all others near at hand perished in the flood. The time is fixed as 11:58 P. M., on March 12, 1927, by the interruption of an electrical power line which crossed just below the dam, and by the fact that the city power plant in the canyon 1½ miles down stream went out of service 5 min. later.

Earlier in the evening many persons passed along the highway below the dam, but not one of them, nor the city power station employees, nor the watchmen on duty, noted anything worthy of report. Had there been material increase in the flow of the very small stream passing down the gulch, it would with certainty have attracted attention and caused an alarm.

Only the day before the break an inspection by officials disclosed to them no signs that indicated danger to the structure. From these facts it is apparent that there was no immediate visible warning and that the break came like an explosion. Nevertheless, there are circumstances which indicate that the dam was in distress for some considerable time before its final rupture.

Several months earlier cracks were noted near each end. These cracks extended diagonally upward from the base and toward the center of the dam. They were open widest at the bottom and were obviously due to stress and not to shrinkage. Movement was evident and it was clearly indicated that the extreme end sections had been loosened and pushed outward and upward by a thrust coming along the arch ring. This force was a transference of load resulting from the deflection or the yielding of parts nearer the center of the dam. The end sections had little resistance at the abutments, due to low coefficient of friction and the sharp angles at the base; they yielded slightly, thus starting the disintegration of the dam.

Other evidence of pre-movement of parts of the structure is established by the record of the recording water-gauge which remained in place on the standing part of the dam. This instrument shows that beginning at least twenty-four hours before the break, there was a gradually increasing change in relative elevation; either a fall in the lake level or an upward movement of the masonry supporting the gauge. There was no water drawn from the reservoir during this time and there could have been no drop in the water level as the change indicated by the gauge would have increased the flow of the creek many times.

It follows that the center section must have lifted slightly during this time, but that was a circumstance to be expected as accompanying partial yielding at the base preceding the final rupture. There were numerous nearly vertical cracks in the masonry caused by shrinkage of the concrete. These contributed largely to the manner of the final breaking up of the mass; but except as they may have allowed a slight sliding of the side-hill sections toward the center, they did not influence the failure. From what is known there can be no doubt that certain parts of this dam were in an unstable condition for some time before the break and that yielding started even before full head was attained.

How the Dam Failed.—The principal weakness of the structure developed on the side slopes where the coefficient of friction (probably not more than 0.5) was altogether inadequate to hold the end sections in place at full head. For a time these sections were supported by the far stronger gravity sections standing in the bottom of the gulch. It is quite significant to note the fact, that had the middle sections of the dam been removed, the ends would have slid down the slope toward the center by reason of their own weight, without requiring any water pressure whatever to start the movement.

When the water in the reservoir reached a depth of about 190 ft. it is probable that there were slight movements in the masonry other than deflections, accompanied by a transference of load through the arch ring. As the head increased, the structure deformed and because of insufficient reaction at the abutments, the ends of the dam were raised and the bond at their base was broken, starting at the top and gradually extending downward. With further increase of head and uplift, the ends yielded more and more and the center, being unable to support the increasing load, leaned forward, raised slightly, and was about ready to fail when it was relieved of a part of its load when the end sections went out.

The movements of the masonry at first were very slow as the mass cracked, weaved, raised, and yielded in places. The final rupture, however, occurred with great suddenness when some section at last squeezed out. When this happened only Sections 1, 2, 3, and 4 (see Fig. 3*) remained erect and, sustaining less load, they settled back nearly to their original positions. The part to the east (Sections 2, 3, and 4) fell later, after its foundation had been undercut by the rushing waters.

This dam failed as any great irregular mass of masonry would be expected to fail under the same conditions. There was first a yielding at points of greatest weakness, then a transference of load to stronger parts, followed by more yielding and further transference of load—all evidenced by cracking and slight movements within the mass. For a time the dam held together as a unit but, in the end, it broke up and went out at the places where it had the least support.

It is important to note that when the parts of the dam slid out of place they fell on their up-stream faces. There was no overturning or the sections would have fallen the opposite way. Just which part squeezed out first it is impossible to say, but evidence points to some section at or near the one marked No. 5 in Fig. 3. Up to the time of final rupture the side-hill sections were crowding down upon the center, the resulting thrust from the east being balanced by a like thrust from the west. However, when some section finally went out the thrust from that side was released and the remaining unbalanced thrust from the other side pushed away from it the adjacent supporting section at the center. This, apparently, is what happened, and, as the east wing slid out, the thrust from the west moved the standing section (No. 1) to the east and left it in its present position. This side movement also freed the west wing, and it immediately went down stream following the east wing by perhaps only a few seconds.

* *Proceedings*, Am. Soc. C. E., October, 1929, Papers and Discussions, p. 2152.

There is nothing mysterious or remarkable about the fact that the center of the dam stood while the ends went out. The mid-section was the stronger, it stood on a level footing, and the resistance to sliding was far greater, due to a better coefficient of friction at this point and to more weight in proportion to the acting head. However, the fact that the central part did not go out does not justify its design; it only demonstrates that the weakest part of a structure will fail first. This part was undoubtedly on the "ragged edge" and would have gone out under a little more head or with a slight increase in uplift.

The element of time as it enters into the conditions of this failure clearly indicates that the forces tending to move this dam were the result of the laws of hydraulics, and not those of hydrostatics as assumed in the old practice of dam design. The forces resulting from the water pressure in this case, lagged behind the increase in head. The penetration of water through rock and masonry is very slow, but until the flow is stabilized the uplift and other pressure conditions continue to increase. The time may be days, weeks, or even years.

Why the Dam Failed.—The St. Francis Dam failed because the reactions at the base in a horizontal direction were less than the forces tending to cause movement.

At or near the base of every gravity dam there exists a definite cleavage plane. A mass of masonry cannot be permanently fastened by cement to the natural rock on which it stands. Such a contact will transmit stress in compression, but not in tension, and hence it is a plane of cleavage. This plane may not be at the actual contact, but if not it will develop at a near-by parting plane in the natural rock. The attachment of a layer of rock to the concrete base, as reported in connection with the St. Francis failure, merely indicates that a plane of greater weakness existed in the rock immediately below. On any parting plane the frictional resistance along that plane in any direction, is the product of the force normal to that plane and a coefficient of friction, which must be experimentally determined.

The force tending to slide a dam on a horizontal base results from the water pressure acting near its vertical face; and for a reservoir dam without overflow, this force increases as the square of the head. For every gravity section there exists a definite relation between the water thrust at the base and the total weight. This ratio, called the sliding factor, is of the same order as the coefficient of friction; the one measures the active force tending to cause movements, the other measures the resistance which holds the structure in place. It follows then, if in any given case the sliding factor is greater than the coefficient of friction, that the mass will slide; if it is less there will be no movement. For any stated head and section on a horizontal base, the sliding factor will vary inversely as the weight of the masonry.

A comparison of the sliding factors at various heads with the coefficient of friction, will show the extent of the zone of safety. For example, if the sliding factor for a given section under certain assumptions as to head, weight, and uplift, is found to be 0.69 and if the coefficient friction for similar surfaces is not more than 0.72, then the margin of safety against all contingencies and errors would be about 4½ per cent. Applied to a section 100 ft. high, such con-

ditions would mean that, with all the other assumptions found to be correct, an addition of 2½ ft. of head would cause failure by sliding. Margins of safety of this order are not uncommon in gravity dams designed in accordance with common practice.

An examination of the St. Francis Dam for sliding factors at various heads, both with and without uplift, is very enlightening. For the maximum section of this dam, Fig. 7 (*a*) illustrates how the sliding factors vary with the head: Curve *A* for the assumed condition of no uplift, Curve *B* for one-half of full uplift at the heel changing uniformly to no uplift at the toe, and Curve *C* for full uplift acting at the heel with other conditions the same. For a side-hill section of this dam of approximately 140 ft. in height, Fig 7 (*b*) illustrates the sliding factors. Fig. 7 (*c*) is for the same section and conditions as Fig. 7 (*b*) except that the base is assumed to have a down-stream slope of about 6%, corresponding to certain of the conditions of the east wing.

On these diagrams will be found marked the probable limits of the coefficients of friction for the several parts of the dam. Under the maximum section near the center, the coefficient of friction was undoubtedly the highest. For illustration this may be taken at the upper limit for such rock as 0.75. For the sections on the side-hill slopes, the coefficient was probably not more than 0.5, and for the schist on its wet cleavage planes it may have been still less. From an inspection of Fig. 7 (*a*), it is apparent that the maximum section of this dam could have resisted full head only so long as but little uplift became effective. With an uplift of less than one-half at the heel, the center must have failed.

On the side-hill sections the conditions were far worse. Referring to Fig. 7 (*b*) and taking the coefficient of friction as 0.5, it is apparent that the end sections must have failed at less than full head even if no uplift was operative. Any uplift whatever would have reduced still further the head at which these sections became unstable. Even had the coefficients of friction been materially higher than 0.5, failures would have occurred with moderate uplift. Undoubtedly parts of this dam started to yield when the water in the reservoir was about 190 ft. deep, and from that time there continued to be yielding followed by transference of load to the final rupture. The conclusion is inevitable that this dam slipped and slid its way to destruction simply because the frictional resistance at the base was entirely inadequate.

The Coefficient of Friction.—The laws of friction are not well understood and all those in use are more or less approximations. There has been surprisingly little experimentation on the friction of rocks in place or for concrete surfaces, and essentially no modern research on this subject.

What is known of the frictional resistance of masonry and rocks has almost entirely come from the Nineteenth Century. Coulomb first enunciated the approximate laws of friction about 1781. Gen. Arthur Morin, a distinguished French engineer, beginning about 1831, made extensive experiments to determine the coefficients of friction for a long list of materials.* Working with pressures of not more than 30 lb. per sq. in., and with very crude equipment,

* "Fundamental Ideas of Mechanics," by Morin, translated by Joseph Bennett, 1860.

he re-affirmed the laws of Coulomb and gave the Engineering Profession the coefficients of friction for rock and masonry in use to-day. Professor W. J. M. Rankine, many years later, in discussing the design of gravity dams, used the tables of Morin, and all other writers have since followed in his footsteps.*

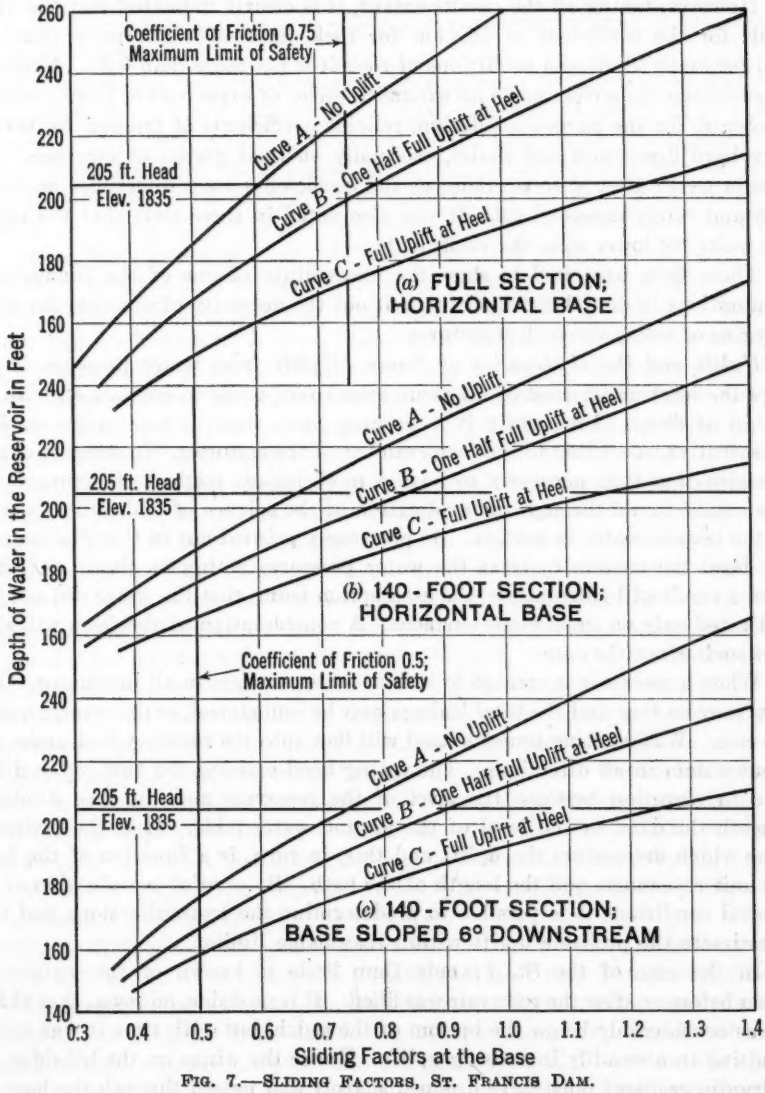


FIG. 7.—SLIDING FACTORS, ST. FRANCIS DAM.

The work of General Morin was a classic at the time; but he did not have the advantage of modern precision apparatus, nor was he able for other reasons to extend his investigations beyond narrow limits of pressure. He did not

* "Applied Mechanics," by Rankine; also, "Miscellaneous Scientific Papers."

examine the common rocks on their cleavage planes, nor did he go into the matter of water lubrication. Concrete as it exists to-day was of course unknown to him. Other investigators have confirmed many of Morin's figures, but there is still a woeful lack of exact knowledge on this subject.

However, taking all the results extant, it is clearly indicated that the upper limit for the coefficient of friction for rock surfaces is not more than 0.75 and for many kinds and conditions of rock it is not more than 0.50. A number of years ago the writer made an extensive series of experiments in an endeavor to obtain, for the purpose of design, reliable coefficients of friction for certain very hard limestones and shales, especially on their planes of cleavage. The results were rather disconcerting as the coefficients were sometimes as low as 0.45 and rarely exceeded 0.6. It was also noted in these tests that the harder the rocks the lower were the coefficients.

These facts are cited to show the uncertainty of one of the fundamental assumptions in dam design and to point out the necessity of allowing far wider margins of safety for such structures.

Uplift and the Hydraulics of Dams.—Uplift from water pressure is perhaps the least understood of all assumptions now made in connection with the design of dams. Frequently it is nothing more than a mere guess—rash or conservative, according to the temperament of the engineer. In order to change this situation it is necessary to have a more logical method of approach and this can be found through the application of the science of hydraulics; that is, by the laws of water in motion. As previously pointed out in this discussion it has been the custom to treat the water pressures acting on dams as if they were a result of hydrostatics—the assumption being that the water did not flow and acted only on impervious surfaces. A consideration of the facts will show that such is not the case.

When a reservoir is created by a dam, water escapes in all directions. This flow may be slow and the total leakage may be immaterial, or the reverse may be the case. Water acting under a head will flow into the masonry and under and about a dam in all directions. The acting head causing the flow is the difference in elevation between the level of the reservoir and the free discharge beneath the dam, or the level of the ground water-table. It is the hydraulic slope which determines the uplift and this, in turn, is a function of the head, the unit resistance, and the length of the path. By careful examination of the natural conditions it is possible to predetermine the hydraulic slope and thus to estimate the probable uplift within reasonable limits.

In the case of the St. Francis Dam little is known of the water-table either before or after the reservoir was filled. It is probable, however, that at first it was considerably below the bottom of the gulch, but with time it was raised, resulting in a steadily increasing uplift. Under the wings on the hillsides, the hydraulic gradient must have dropped sharply and passed through the base far in from the toe, resulting in only little uplift.

To say that no uplift exists under a dam is to make an absurd statement and one contrary to all scientific knowledge. The uplift may be great or small depending on the hydraulic conditions, but to some extent it will always

exist and it will frequently happen that the better the rock is for foundation purposes, the greater will be the resulting uplift. In the absence of positive information to the contrary, full uplift at the heel to zero at the toe would appear to be the only permissible assumption. Had this practice been followed the St. Francis Dam would have been of greatly enlarged section. The same remark applies with equal force to many dams now standing, and to others under consideration.

The proposed San Gabriel Dam near Los Angeles, Calif., 500 ft. in height, as designed with gravity section, has a margin of safety of less than 10% at half uplift at the heel, and no margin at all at full uplift. The proposed Colorado River Dam, 700 ft. in height, with the section used for the estimate, has a still less margin of safety at half uplift and is certainly unstable with full uplift in action. These samples illustrate prevailing conditions. Gravity dams can be built, however, with adequate safety and at reasonable cost, but as at present designed they are an unjustifiable hazard to life and property.

Leakage and Seepage.—Leakage and seepage are not synonymous terms although they are frequently so used. Leakage is the wastage of water occurring through cracks in the masonry or through joints, faults, or other open passages in the rock formation. Generally speaking it has little to do with the forces acting on a dam, and it becomes a source of danger only when the velocity of discharge is so great as to cause scour.

Seepage, on the other hand, is the passage of water through normal interstices of the rock and masonry. Practically all rocks are permeable in some degree and concrete is especially so as usually made. Thus, it is to be expected that when water head is applied above a dam, seepage will develop. In amount it may be so small and so generally distributed as to be almost invisible. Nevertheless, it is there, and it continues to increase with time until for a given head a uniform flow is established. Seepage cannot be prevented altogether in a natural rock formation and it can be checked only on the plane where the water enters. Leakage on the other hand can be sealed off by well-known methods. Cement grouting is often very effective in closing leaks, but it fails to prevent seepage through a porous rock. Cracks and small openings in a dam are objectionable for the reason that they collect and redistribute seepage and they should be reduced to a minimum. However, the entire absence of visible leakage in no way indicates that the structure and the rock below are not being subjected to heavy hydraulic pressures producing what is commonly called uplift.

Artificial drainage of the rock has been frequently advocated as a means of reducing uplift. It is, however, most ineffectual and may become an added source of danger in high dams. Drainage passages do not collect seepage for any appreciable distance from their walls and act only as a means of discharging leakage water coming to them through cracks in the concrete or open passages in the rock. For high-head dams a reasonably long watercourse through the bed-rock is desirable as a matter of safety. In such cases the spouting velocity at full head is very high and any arrangement that concentrates the full pressure of the water on a narrow band of rock will add greatly to the hazard.

The St. Francis Dam as built had only a very limited drainage and this feature in no way changed the result. The lesson is that seepage with its resulting uplift was a contributing cause to this failure.

The Middle Third Theory.—Implicit faith in the infallibility of the so-called "middle third theory" appears to have been one of the indirect causes leading up to the St. Francis Dam failure.

The authorship of this so-called theory is usually attributed to Professor Rankine.* However, an examination of his writings discloses the fact that he never proposed it as a comprehensive theory or method of design, but rather as a precautionary limitation intended only for the guidance of the non-technical engineers of his day. This suggestion grew out of elemental geometrical and mechanical conceptions and was one of several made by him for conditions then prevailing. Empirical expressions and methods were much in vogue at the time and certain "general rules" as proposed by Rankine for limited phases of design, have since been extended by other writers until the "accepted theory" has become a kind of engineering fetish followed by many designers without reference to the fundamentals of modern science or to the changed conditions arising from the new materials and the far greater dimensions of present-day structures.

The requirement that the resultant of all the forces acting shall fall within the middle third of horizontal sections is only one elementary test and in no way assures the safety of this structure in other respects. Of this requirement Professor Rankine has well stated, "It appears that the exact determination of the limiting position of the center of pressure at a plane joint is, strictly speaking, a question relating to the strength of materials".†

The best position for the resultant from a structural standpoint is at the middle point of the base, as this will produce the most uniform loading on the foundation. It is permitted by design to fall at some other point in order to save material. This is a proper economic procedure if it does not reduce the margin of safety to an unreasonable extent. In other words it is measured by the risk allowable for such structures. It is obviously possible and rather common for dams designed in conformity with the "middle third theory" to have little or no margin of safety. This was the case with the St. Francis Dam and should be sufficient reason for discarding in the future such a defective practice in design.

Curved Dams and Arch Dams.—Not all dams curved in plan are arched dams, although they are often so considered. Within conservative limits of width and in locations having adequate natural abutments, properly designed arched dams are both safe and economical structures. However, they are not under discussion here. When the width is too great for an arched dam recourse must be had to some form of dam supported at the base. The type in most common use for this contingency and the one here under discussion, is the solid gravity dam of concrete masonry.

* See "Miscellaneous Scientific Papers"—"Report on the Design and Construction of Masonry Dams," by Rankine; See, also, "Scientific Papers of Rankine," "Applied Mechanics," and "A Manual of Civil Engineering."

† "Applied Mechanics," by Rankine, p. 227.

Some designers have thought that the curvature of a gravity dam added something to its strength even at sites where adequate arch support was an impossibility. In cases where full load cannot be carried by a properly designed arch it is very questionable practice to endeavor to support a part by arch action and the remainder by gravity. Structural designers in other lines have long recognized the dangers involved in an attempt to sustain a structure by two radically different systems of support. To endeavor to strengthen a deficient gravity dam by means of a weak arch is at least unsound engineering. The difficulties of design in this direction are not alone those of mathematical computation; but to a far greater extent they are the unknown physical qualities of the concrete and of the natural rock in the foundations and abutments.

The writer holds strongly to the opinion that in the present state of knowledge, it is most unwise to endeavor to support a dam by a combination of arch and gravity reactions. To be entirely secure the structure should be one thing or the other, a gravity dam, or an arch dam, in either case with ample margin of safety. The St. Francis Dam was a very weak structure as a gravity section, and it had little or no strength as an arch. The curvature was a defect and a straight dam utilizing the same amount of material would have been measurably stronger.

Reducing the Hazard.—If the lessons of the St. Francis Dam are to be heeded in the future, there must be many changes in the methods of designing and building gravity dams. A few of the changes vitally necessary as they appear to the writer, are as follows:

1.—Far wider margins of safety must be allowed, and these should be clearly expressed and made applicable to all the conditions and phases of designs.

2.—Gravity dams should be built with bases sloping steeply toward the heel, in order to reduce the tendency to slide on the rock contact.

3.—Friction as a means of sustaining a dam should be relied upon only to the extent essential for economy and every precaution should be taken to ascertain in advance the coefficient of friction applicable to the surfaces in contact.

4.—Uplift and all distribution of pressure arising from the water head, should be determined by reference to the laws of hydraulics. Research in this direction is essential.

5.—Gravity dams should be built in separate monolithic sections of moderate length, reinforced with steel where necessary.

6.—The slopes of the canyon walls should be excavated to such ample depth, that the rock at the base cannot slip from beneath the structure.

7.—Gravity dams should be straight in plan unless the topography makes it necessary to build them in some other form.

8.—At full water head the distribution of load on the base should be as nearly uniform as possible, and this becomes more essential with higher dams and greater unit stress.

9.—The design should be such that there can be no unduly rapid rate of increase in stress, as a result of small additions of head or from slight variations in any of the assumptions.

10.—Ample provision against sliding, together with proper limitations of the stress in the concrete and the rock, are the fundamental requirements for the structural safety of gravity dams.

F. B. MARSH,* M. AM. SOC. C. E. (by letter).†—Doubtless it was difficult to get accurate complete data on the relation to its original position of the section of the St. Francis Dam that remained standing after the catastrophe. Such record as has been published seems to show that the top of the standing section was left about 6 in. down stream and 6 in. to the east of its original position, probably due to tilting. One cannot but wonder how that could have happened if, as many investigators seem to think, the west end of the dam failed first.

Study of the data presented by the Committee and in the engineering press raises the question of whether the initial failure that started the almost instantaneous collapse of the whole structure may not have occurred near the east end, at the sections marked 12 and 5 on Fig. 3.‡

It has been pointed out§ that the cross-section of this dam was not adequate at the ends as a gravity structure if hydrostatic uplift occurred under the base. The laminated mica schist under the east end of the dam made an ideal condition for the transmission of hydrostatic pressure in large areas, not only under the base but also through the seams of the rock well below the base. Such pressure might easily have burst out suddenly through the soft rock at the down-stream toe of the dam. This, in turn, would have tended to allow Block 5 (see profile, Fig. 3) to move down stream with the pressure. At least one of the diagonal cracks at the ends of this block is said to have existed long before the failure and, since the temperature was cold, any existing cracks were probably opened to a maximum.

The incipient movement of Block 5 would undoubtedly have started a landslide at that end of the dam even if none had started before. Inspection of the plan of the dam (see Fig. 1‖), shows that immediately up stream from the east end the contours of the original material were nearly parallel to the up-stream face. The filling of the reservoir or the movement of Block 5 may have started this material sliding toward the dam. Such a saturated mass of earth and rock would exert a much greater thrust against the dam than water alone and might well have caused the breaking up of the portion between Block 5 and Block 1, which was left standing.

The presence of the great mass of this easterly landslide would have delayed, momentarily, the rush of water through the easterly gap. The longitudinal thrust from the westerly end, itself already weakened by under-seepage, might have been sufficient to raise the westerly edge of the block that was left standing and tip its top down stream and toward the east where its support was already gone, spalling off the down-stream toe at the same time. This movement, although slight, would have been enough to let the westerly portion blow out, after which the portion left standing would have fallen back, but not quite to its original position.

* With Board of Water Supply, New York, N. Y.
† Received by the Secretary, January 8, 1930.
‡ *Proceedings*, Am. Soc. C. E., October, 1929, Papers and Discussions, p. 2152.
§ *Loc. cit.*, January, 1930, Papers and Discussions, pp. 163 and 172.
‖ *Loc. cit.*, October, 1929, Papers and Discussions, p. 2149.

F. C. FINKLE,* ESQ. (by letter).†—In the summer and fall of 1924, the writer was retained by the Anti-Annexation Committee of the City of Santa Monica, Calif., to report on the desirability of the City of Santa Monica becoming annexed to the City of Los Angeles. This required an examination of the Los Angeles-Owens River Aqueduct, including construction work theretofore completed and then in progress on this project. A report on the conclusions reached was submitted under date of September 8, 1924, and the writer delivered some public addresses in line with his report during the annexation campaign which followed.

The examination on which the report was made occurred August 2, 1924, when the excavation at the site of the St. Francis Dam was practically completed. Fourteen samples of the foundation material in the trench for the St. Francis Dam were gathered and subjected to the water test. They dissolved in from 1 to 10 days; no mechanical tests were deemed necessary. The report called attention to the defective plans for the dam by showing that the base thickness was as low as 63% of the height in some sections, and was about 85% of the height in the middle section; it also expressed the opinion that even a thickness of base equal to the full height might not be safe on such a foundation, and suggested that some type of elastic dam should be designed for a site of that nature.

The subject was much discussed during the annexation campaign which ensued, but those favoring annexation to Los Angeles failed to be convinced that the St. Francis Dam was unsafe, even after the election went against them. The fact that doubt about the safety of the St. Francis Dam as well as of the Mulholland Dam had been raised in the minds of the people of Santa Monica no doubt had much to do with the overwhelming defeat of the proposition. The fact, pointed out by the writer, that the total supply for the Los Angeles Aqueduct from Owens River that year (1924) was little more than a mean of 200 cu. ft. per sec., or about one-half the capacity of the aqueduct, of which more than one-third was pumped from the ground-water supply in Owens Valley, also had much influence on the result.

When the writer next visited the St. Francis Dam site, on August 27, 1927, the dam had been in use for some time. Although the reservoir was not full, the leakage amounted to about 6 sec-ft. There were several horizontal cracks in the hillside of each abutment, which were too small to be visible, except as they were evidenced by seepage of water along horizontal lines, extending from the ends of the dam into the hillsides. The danger was so obvious that the writer left the canyon rather hurriedly, fearful that the dam might fail at any time. He expressed his fears to the man in charge of the power house below the dam, who merely laughed at the suggestion of possible danger. The dam failed in March, 1928.

In the writer's opinion, two causes contributed to the failure of the St. Francis Dam, as follows:

1.—The design of the two end sections was defective, because it did not provide a sufficient factor of safety against hydraulic uplift, acting con-

* Cons. Engr., Los Angeles, Calif.

† Received by the Secretary, February 1, 1930.

jointly with other stresses, when the reservoir was full of water. This defect in design would in time have caused the dam to fail, even had the foundation material been much better than it was.

2.—The foundation under the dam, especially under the west end, became so soft that it was being displaced and squeezed out to some degree. This caused the rigid dam to settle, producing the disruption of the adjacent hillsides and the fine horizontal cracks, evidenced by fine leakage along nearly horizontal lines.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

ADJUSTMENT OF TRANSIT AND STADIA TRAVERSES

Discussion*

By Messrs. Paul H. Underwood, F. H. Peters, and H. Parry.

Paul H. Underwood,† M. Am. Soc. C. E. (by letter).‡—The author has well assumed§ as a basis for the application of the method of least squares to this problem that the corrections applied to the latitudes and departures should not affect the azimuths or bearings of the lines of the traverse, and the method devised seems to be as simple as any that can be used for this purpose.

It has occurred to the writer, however, that an even better distribution of error, from the theoretical standpoint, is obtained by considering the corrections to vary as the lengths of the lines of the traverse. This may be done by introducing weights for the various courses. If these weights are considered to vary inversely as the lengths, the solution can be carried out with no more labor than by Mr. Rappleye's method. The necessary changes in his equations are indicated as follows.

The latitude of a course is designated by L, and the departure by D. Furthermore, let $\Sigma\ (L \cos a) = A'$; $\Sigma\ (D \cos a) = B'$; and $\Sigma\ (D \sin a) = C'$.

With the weights varying inversely as the lengths of the courses the following equations may be derived to replace Equations (3)‖ and (4)‖, respectively:

$$A'\ M_c' + B'\ M_s' + F_L = 0 \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

and,

$$B'\ M_c' + C'\ M_s' + F_D = 0 \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

The solution of these equations for M_c' and M_s' gives:

$$M_c' = \frac{B'\ F_D - C'\ F_L}{A'\ C' - (B')^2} \ldots\ldots\ldots\ldots\ldots\ldots (10)$$

* Discussion on the paper by Howard S. Rappleye, Assoc. M. Am. Soc. C. E., continued from January, 1930, *Proceedings*.

† Prof. of Surveying, School of Civil Eng., Cornell Univ., Ithaca, N. Y.

‡ Received by the Secretary, January 4, 1930.

§ *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2302.

‖ *Loc. cit.*, p. 2303.

and,

$$M_s' = \frac{B'\,F_L - A'\,F_D}{A'\,C' - (B')^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

The desired corrections to the observed lengths are, therefore:

$$\left.\begin{aligned} V_1 &= M_c'\,L_1 + M_s'\,D_1 \\ V_2 &= M_c'\,L_2 + M_s'\,D_2 \\ V_n &= M_c'\,L_n + M_s'\,D_n \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (12)$$

To illustrate this method, Table 2 shows the computations of the same traverse for which the calculations by the author's method were shown in Table 1.* In the first eight columns, that is, to and including Column (7), the tabulations are the same for the two methods, and are not repeated in Table 2. The quantities in Column (8), (9), and (10), Table 2, were computed by the use of the ordinary 10-in. slide-rule. It may be noted that all the signs of all the quantities in Columns (8) and (10) must necessarily be positive. The footings of the three columns are, respectively, A', B', and C'. Substitution in Equations (10) and (11) gives -0.001104 and $+0.001543$, respectively, as values of M_c' and M_s'. By use of the slide-rule these values are multiplied by the latitudes and the departures, respectively, the calculated values being entered in Columns (11) and (12). The computations from that point are similar to those in Table 1.

TABLE 2.—COMPUTATIONS FOR CLOSING THE EIGHT-SIDED FIELD SHOWN IN FIG. 1.*

Course.		$L \cos a$.	$D \cos a$.	$D \sin a$.	$M_c' L$.	$M_s' D$.	$M_c' L + M_s' D$.	Correction to observed distance, in feet.	Adjusted length, in feet.	Adjusted.	
No.	Station to station.									Latitudes.	Departures.
(1)	1(a)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
1	A–B	+ 722	+240	+ 80	+0.84	−0.39	+0.45	+0.4	802.4	−761.2	− 253.7
2	B–C	+ 360	+360	+ 360	+0.56	−0.79	−0.23	−0.2	719.8	−509.0	− 509.0
3	C–D	+ 722	−587	+ 482	+1.03	+1.17	+2.20	+2.2	1 204.2	−932.0	+ 762.4
4	D–E	+ 28	+167	+1002	−0.19	+1.57	+1.38	+1.4	1 030.4	+169.5	+1016.4
5	E–F	+ 570	+162	+ 46	−0.65	+0.26	−0.39	−0.4	614.6	+590.8	+ 168.9
6	F–G	+ 662	+165	+ 41	−0.75	+0.26	−0.49	−0.5	700.5	+679.6	+ 169.9
7	G–H	+ 847	−386	+ 196	−1.03	−0.66	−1.69	−1.7	1 023.3	+931.5	− 423.5
8	H–A	+ 30	+167	+ 917	+0.19	−1.54	−1.35	−1.4	946.6	−169.2	− 931.4
Totals.	.....	+3941	+288	+3124	.....	.....	.....	.....	.....	0.0	0.0

* *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2302. As stated in the text, Columns (2) to (7), inclusive, of the author's Table 1, have been omitted in Table 2.

A comparison of results by the two methods shows in this case no very great differences. One reason is that the courses do not vary markedly in

* *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2304.

length. A close examination reveals, as might be expected, that the corrections to the longer lines are in general larger and those to the lengths of the shorter lines are smaller by the writer's method than by that of the author. To get a comparison which would indicate more clearly the variation in the results by the two methods, the writer has taken another traverse in which there is a greater variation in the lengths of the courses. The resulting corrections are shown in Table 3.

TABLE 3.—COMPARISON OF CLOSURE CORRECTIONS FOR FIELD CONTAINING SHORT AND LONG SIDES.

Course.		Bearing.	Distance, in feet.	Correction to observed distance, in feet, by author's method.	Correction to observed distance, in feet, by writer's method.
No.	Station to station.				
1	*K-L*	N. 23° 55′ E.	528	−0.3	−0.2
2	*L-M*	N. 38° 36′ W.	540	−2.1	−1.7
3	*M-N*	N. 50° 43′ E.	671	+0.7	+0.7
4	*N-O*	N. 78° 51′ E.	375	+1.6	+0.9
5	*O-P*	S. 20° 25′ E.	868	+1.8	+2.3
6	*P-Q*	S. 19° 38′ W.	447	+0.5	+0.3
7	*Q-K*	S. 79° 53′ W.	937	−1.6	−2.3

It will be noted that the variations in the corrections are more apparent, the writer's method, based on weights, giving for the shorter courses smaller corrections and for the longer courses larger corrections. This is paticularly noticeable when Courses Nos. 4 and 7 are compared, the method without weights leading to equal corrections for these two courses, while the method with weights gives a correction of +0.9 ft. for the shorter course and −2.3 ft. for the longer one. Since these two courses are nearly parallel, the variation is not due to other factors to any extent, but is due almost entirely to the weighting.

In conclusion, it may be said that, where all the courses are very nearly of the same length, there will be little variation in results by the two methods. Where there is considerable variation, the results will be noticeably different. Since the calculations are no more lengthy or difficult by the method with weights, and since this gives results which, on theoretical grounds, are superior, it would seem that this method is preferable.

F. H. PETERS,* M. AM. SOC. C. E. (by letter).†—This paper deals with a problem that has been the subject of special investigation by the Topographical Survey of Canada, which has probably done more of this class of work than any other survey organization.

The rapid development of Canada's northland, immediately following the World War, hastened by the discovery of rich mines and a realization of the vast potential mineral wealth of the Pre-Cambrian shield, made a sudden demand on this Survey for topographical maps of hitherto unexplored country, and for surveys through this country to permit of the proper registration

* Surveyor General, Dept. of the Interior, Ottawa, Ont., Canada.
† Received by the Secretary, February 28, 1930.

of all claims staked. The area to be surveyed is so vast that chained control surveys were out of the question, from considerations of speed as well as of cost. Hence, stadia traverses were run along the principal waterways for the dual purposes of providing a skeleton network of control surveys for the tying in of mineral claims and for ground control for the oblique aerial photographs from which the country was to be mapped.

In order to control these extensive stadia surveys, they were arranged in closed circuits wherever possible, and where this was not possible, astronomical observations for latitude and longitude were taken at intervals of 75 to 100 miles, and the stadia surveys were adjusted between observation points. The extent of these operations may be judged from the fact that in the six years immediately following the war, no less than 25 000 miles of stadia traverse were run by members of the Topographical Survey of Canada.

As regards the field work, it may be said that every precaution was taken to ensure freedom from gross errors both in directions and in distances, and many novel features were incorporated into the work with this end in view. Briefly, angle measurements, checked by frequent star observations for azimuth, are seldom more than 2′ or 3′ of arc in error, but the distances are subject to the inherent weakness of stadia measurements, so that as Mr. Rappleye rightly states,* "the angle measurements far surpass the distance measurements in the matter of accuracy."

The field closing stipulated as being permissible between control points is $\frac{1}{400}$ of the surveyed distance, and these closing errors had to be distributed in some way. This led to an investigation of the method of adjustment best adapted to the conditions of the survey, while yet not entailing too much office work. The tremendous amount of computations involved had a very important bearing on the method of adjustment to be adopted.

The investigation† was made by H. Parry, Mathematician of the Topographical Survey of Canada, who first examined the advantages and defects of the well-known Transit Rule‡ for balancing a closed survey. It was found that this rule produced appreciable changes in the accepted bearings of the courses, which were determined in the field with considerable accuracy. Consequently, an investigation was made to deduce a rule which would distribute the closing errors by correcting the lengths while leaving the bearings unchanged. This is exactly the same problem as that outlined by Mr. Rappleye, a Least Square adjustment of which was made with exactly the same results as obtained by him.*

After some consideration, however, it was decided that this Least Square adjustment, while possessing the advantage of leaving the bearings unchanged, was not exactly suited to the field conditions under which Canadian Government surveys are run. It is based on the assumption that all the errors in the lengths are of the accidental class, while actually the lengths are also affected by systematic or constant errors which it is not possible to eliminate entirely.

* *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2301.

† The complete mathematical investigation referred to is on file in the Engineering Societies Library, 33 West 39th Street, New York, N. Y.

‡ "Theory and Practice of Surveying," by J. B. Johnson and L. S. Smith, Seventeenth Edition, 1912, pp. 235–236.

A study of these systematic errors shows that a certain proportion may be accounted for by variation in the stadia constant of the instrument with different climatic conditions. This is due partly to an actual variation in the distance apart of the stadia threads and partly to the consistent bending of the rays of light, especially the lower ray, under certain atmospheric conditions. While engaged on work of this nature in Canada, field surveyors of the Topographical Survey of Canada, are instructed to determine the stadia constant of their instrument from time to time during the course of the survey and where necessary, corrections are applied to the stadia lengths. Glass diaphragms are found to give a less variable stadia constant than spider threads. It is not possible, however, to correct for the systematic errors due to refraction and other minor causes.

In addition to these considerations, the Least Square adjustment, as applied to the extensive traverses of the Topographical Survey of Canada, would entail an enormous amount of computation, and this led to the selection of the much simpler Transit Rule. Incidentally, it might be mentioned that the Least Square adjustment necessitates corrections to the lengths which, in general, are greater than those given by the Transit Rule. This, of course, is obvious, since the Transit Rule besides changing the lengths, also changes the bearings up to as much as 8′, while the Least Square adjustment confines all the corrections to the lengths.

It will be seen, therefore, that the Least Square adjustment deduced by the author would not suit the field conditions encountered by the Topographical Survey of Canada. It can readily be understood, however, that entirely different conditions of field and office work might make it very desirable to adopt a Least Square adjustment, and the Surveying Profession is indebted to Mr. Rappleye for bringing this particular solution to its notice.

H. PARRY,* ESQ. (by letter).†—In 1921 the writer made a study of the methods of adjusting stadia traverses in order to distribute the closing errors of the extensive stadia traverses of the Topographical Survey of Canada. As the Transit Rule‡ is by far the most widely known and used rule for adjusting and balancing traverse surveys, the following investigation into the effect of this rule on the bearings of the traverse will be of interest in connection with Mr. Rappleye's paper.

The rule has been formally expressed by Johnson and Smith, as follows:

"As the arithmetical sum of all the latitudes is to any one latitude, so is the whole error in latitude to the correction to the corresponding latitude, each correction being so applied as to diminish the whole error in each case. Proceed similarly with the departures."

To evaluate the effect on the bearing of any particular course, let:

l = measured length of any particular course.
a = bearing of course.
L = arithmetical sum of the latitudes.

* Mathematician, The Topographical Survey of Canada, Ottawa, Ont., Canada.
† Received by the Secretary, February 28, 1930.

‡ "Theory and Practice of Surveying," by J. B. Johnson and L. S. Smith, Seventeenth Edition, 1912, pp. 235–236, and other standard books on surveying.

D = arithmetical sum of the departures.
F_L = error of closure in latitude.
F_D = error of closure in departure.
λ = correction to latitude of course considered.
μ = correction to departure of course considered.
δa = change in bearing of course considered caused by a correction of F_L to latitude and F_D to departure.

Stated algebraically, the Transit Rule for any particular course gives, for latitude:

$$\frac{\lambda}{l \cos a} = \frac{F_L}{L} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

and, for longitude:

$$\frac{\mu}{L \sin a} = \frac{F_D}{D} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (14)$$

The corrected latitudes and departure of this course are $l \cos a + \lambda$ and $l \sin a + \mu$, respectively. Hence,

$$\tan (a + \delta a) = \frac{l \sin a + \mu}{l \cos a + \lambda} = \tan a \left\{ \frac{1 + \frac{\mu}{l \sin a}}{1 + \frac{\lambda}{l \cos a}} \right\}$$

$$= \tan a \left\{ 1 + \frac{F_D}{D} - \frac{F_L}{L} + \ldots \right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (15)$$

in which, terms of the second order are neglected.

Equation (15) is in the well-known form, $\tan x = \rho \tan y$, the solution of which is, $x - y = \frac{\rho - 1}{\rho + 1} \sin 2y + \ldots$. Hence:

$$\delta a = \frac{\frac{F_D}{D} - \frac{F_L}{L}}{2 + \frac{F_D}{D} - \frac{F_L}{L}} \cdot \sin 2a + \ldots\ldots\ldots\ldots\ldots\ldots (16)$$

or, neglecting terms of the second order:

$$\delta a = \frac{1}{2}\left(\frac{F_D}{D} - \frac{F_L}{L}\right) \sin 2a \quad \ldots\ldots\ldots\ldots\ldots\ldots (17)$$

This is the change in bearing of a course of latitude, L, and departure, D, when the latitude is corrected by F_L and the departure by F_D.

In the stadia surveys of the Topographical Survey of Canada, the work is required to close with an error of less than 1 in 400. If this error of closure is due entirely to errors in the lengths, then the correction in latitude to any individual course must be less than 1 in 400 of the latitude of that course and the correction in departure must be less than 1 in 400 of the departure of that course.

This is not strictly true in practice because, in a long traverse running almost due north and south, in which the arithmetical sum of the departures is very small compared with the extent of the survey, the error of closing in departure due solely to small errors of direction might be of the order of

1 in 400. A similar condition would hold for a traverse running almost due east and west. However, in the general case the errors in direction are of a smaller order than the errors in lengths.

In the extreme case, assuming that latitudes are too long by 1 in 400, and departures are short by 1 in 400, or *vice versa*,

$$\frac{F_L}{L} = \frac{F_D}{D} = \pm \frac{1}{400}$$

and, by Equation (17),

$$\delta a = \pm \frac{1}{2}\left(\frac{1}{400} + \frac{1}{400}\right) \sin 2a = \pm \frac{1}{400} \sin 2a$$

or, in minutes of arc:

$$\delta a = 8.5 \sin 2a \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (18)$$

This is the change in bearing in the extreme case where latitudes and departures have their maximum allowable corrections of 1 in 400. It is a maximum for a course inclined at 45° to the meridian, in which case, $\delta a = \pm 8.5'$.

This introduction of errors into traverse bearings which have been accurately determined in the field is, then, the price that must be paid for ease and simplicity in the computations. It may be stated, however, that the work of the Topographical Survey of Canada is seldom affected by errors as large as 1 in 400, which are considered the maximum errors permitted without further investigation of the field work.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

FACTORS GOVERNING THE LOCATION OF AIRPORTS

Discussion*

By Messrs. N. H. Angell, J. Ogden Marsh, Archer R. Norcross, John Nolen, S. N. Grimm, R. U. St. John, and John F. Partridge

N. H. Angell,† Esq.—In this paper a rating of 14 points was given to the physical features of the airport.‡ The factors which would influence the percentage of rating under this classification would evidently be the condition of the surface of the field, and the manner in which the field is laid out. The latter has been well covered in *Bulletins* of the U. S. Department of Commerce and by the author.

The question of surfacing has usually been left to the individual fields to work out for themselves. It is a question of special interest on the Pacific Coast. There is no doubt that some kind of a surfacing is necessary. Due to the lack of rainfall in summer months and the consequent cost of irrigation, turf fields have been virtually prohibited in the Far West. The development of some other type has been necessary.

Aside from the question of elimination of dust, the functions most desired in runway treatment are that it shall provide a firm surface over which planes can accelerate to the desired flying speed in the shortest distance, and that this be done with comfort to passengers at all times when weather permits flying. With the flying speed of planes ranging between 50 and 75 miles per hour, the take-off can be made in a shorter distance when the run is over a firm, even surface than when it is attempted from a rough, irregular, and perhaps muddy field.

Increased passenger carrying is demanding surfaced fields. In the larger airports constructed by aviation corporations which have sufficient money avail-

* Discussion of the paper by Donald M. Baker, M. Am. Soc. C. E., continued from February, 1930, *Proceedings*.

† San Francisco, Calif.

‡ *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2316.

able, the surfacing of the field is one of the major items considered, and in many cases the field is not opened until such surfacing has been completed.

The use of hard surfaces on landing areas is not regarded with favor by most pilots. A comparatively soft, resilient surface is preferred, one that will have an appreciable tractive resistance and aid in slowing down the plane, as well as help to absorb the shock of impact. The drag of a tail-skid on a surface of the proper consistency aids materially in decreasing the run after landing, although its primary function after the plane is on the ground is to exert a stabilizing effect and eliminate any tendency of the plane to veer from one side to the other or "ground loop." In order to insure this desired stabilizing effect, a somewhat soft or plastic landing surface is preferred. Properly constructed, this surface is also suitable for take-offs.

Asphaltic oils with a properly graded sand have been used for the construction of such surfaces. The treatment may consist of a surface application in which the oil is allowed to penetrate, or if more money is available, analysis of the soil may be made and, with the addition of foreign sand to supply any deficiency in the grading of the natural soil of the airport, an oiled sand mat may be built up by successive applications of oil, each application being mixed with the sand by harrowing and blading. This is similar to the "mixed-in-place" method used in the construction of secondary highways in many of the States, except that the grading of the aggregate is quite different. When practical, it is preferable to mix the surface in an asphalt plant in much the same manner as the ordinary asphaltic concrete or sheet asphalt types of surfacing are mixed. This provides for an accurate control of the aggregate and oil at all times, insuring uniformity of the surface.

On the United Airport at Burbank, near Los Angeles, Calif., there are approximately 70 acres of runway surface treated with a pre-mixed oiled sand material. For this work the aggregate was in some cases as large as ½ in., grading down to 200-mesh material. There are three runways on this airport, each 300 ft. wide and averaging 3 500 ft. in length. The plant-mixed asphaltic surface is soft enough to be marked and even creased by tail-skids, but firm enough for take-offs under all weather conditions encountered. In addition, there are about 50 000 sq. yd. of asphaltic concrete, 4 in. and 3 in. in thickness, on roadways, warming-up platforms, and parking areas.

There are a great many airports in California that have oil-treated surfaces and the dust nuisance has been largely eliminated. Such treatment has resulted in increased revenue from passenger carrying and decreased maintenance costs on the airport surfaces and on equipment operated from the field.

Many fields have started in their surfacing program by paving the areas adjacent to the hangars, such as the aprons and warming-up platforms. The pavement has been made as this as possible, consistent with the subgrade and the load to which it will be subjected. By using a minimum thickness, larger paved areas are possible from the same funds. It is felt that if there is an increase in the size of planes to such an extent that the pavement will not be strong enough, bituminous types can be easily thickened by a resurfacing course, without sacrificing the initial investment.

J. OGDEN MARSH,* M. AM. SOC. C. E.—When automobiles first began to appear, most people regarded them as a "rich man's toy", and not until the public took them seriously and as a commonplace part of daily life did the automobile industry begin to grow. The present state of the public mind in regard to aeronautics is somewhat similar, and in the speaker's opinion, there is no more important thing than to have the public become so familiar with airplanes that they will look upon them as ordinary functions of daily life.

The author has stressed the necessity of public support for the acquisition and construction of principal airports†. The public does not expect steamship companies to pay for harbor development, and they probably should not expect air transportation to provide the airports. If aviation interests are to have public support financially, they must make the public "air-minded".

As airplane activities must naturally be centered about airports, the problem resolves itself into one of getting the public to take more interest in visiting airports, both public and private, and in actually seeing the operation of flying. To accomplish this, planning engineers must make airports not only extremely accessible, but they must make them attractive. There must be correlated with airports, and their surroundings, other and more familiar lines of interest and amusement that will bring people to these localities. This is a matter which should be given some thought, because it is important to get the crowds there. To do this with convenience, the necessary roads must be built for motor vehicles.

The desire, however, to attract crowds should prevent the absolute prohibition of stunt flying, as such flying fosters the public belief that airplanes are not a safe and dependable means of transportation.

ARCHER R. NORCROSS,‡ ASSOC. M. AM. SOC. C. E.—The author mentions the layout and construction of runways and gives several types of surfaces which have been advocated for runway construction, or have been already used on some airports. The proper surfacing of the airport is a problem which is being given constantly increasing attention and thought on the part of airport designers and constructors. While the surfacing must provide for an entirely new type of traffic, the methods used by highway engineers, and the lessons which they have learned in the development of durable and economical surfaces, may be well applied to airport work.

Runway surfaces must be firm enough to resist displacement under loads which are already up to 25 tons and may become greater. They must be free from any tendency to rut or break. In addition to the static load which they must stand, the force of impact must be considered. It would thus appear that the surfacing material should be of a type which is easily thickened to take care of the increased loading that may be expected as a result of the tendency for planes to develop along a larger scale.

* Chf. Engr. and Gen. Mgr., Board of Public Utilities and Transportation, Los Angeles, Calif.

† *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2308.

‡ Care, Am. Bitumuls Co., Boston, Mass.

The methods of the highway engineer as exemplified on the new construction work on secondary roads in the West may well be adapted to preliminary construction on runways. The application of an asphaltic oil with surfacing of fine screenings and the whole compacted by rolling so that a water-proof and firm surface is obtained, or some similar method of treatment, will not only provide a runway surface that will serve for some time, but it assures an excellent sub-grade for future paving.

There is one item which enters into the construction of airport surfaces with much more importance than is attached to it in highway work, that is, the visibility of the surface. Oiled or dark bituminous surfaces appear to be regarded with most favor by pilots. It might be of interest to note here that at the request of pilots, all light-colored roadway surfaces in the vicinity of the North Island Naval Air Station, at San Diego, Calif., have been given a color coat of dark material.

The question of visibility at night is entirely one of adequate lighting. Wide markings on dark-colored pavements, or the outlining of such pavements in white, in conjunction with proper lighting equipment, would appear to provide the best visibility both for day and night flying.

A resilient surface aids materially in absorbing sound and impact, both of which are important considerations for airports. With these ideas in mind, it is not difficult to appreciate the trend toward the bituminous type of surfaces on the Pacific Coast.

JOHN NOLEN,* M. AM. SOC. C. E. (by letter).†—In discussing this paper the writer would like to emphasize some of the points mentioned by Mr. Baker, but which owing to the broad scope of the subject had to be limited in consideration. These points are those having city planning relationship. As an outstanding feature of modern transportation the airport has an effect upon the city or urban community as a unit. To choose a site without consideration of all the elements of the community composition may mean that either the city may be injured by the location given over to the airport, or, in turn, the airport may not be so situated as to serve the city economically; or still worse, it may be so placed that it cannot develop business either from the city or serve as an adequate and safe stopping point on an airway for traffic from outside.

In these days American cities and communities of various types have come to a fairly complete comprehension of what city planning means. They have arrived at this comprehension through two channels: (*a*) The necessity of looking after the physical side of the city to meet the ever-increasing congestion due to the increased measure of transportation facilities, the growing density of population, and the recognized need for wider means of recreation; and (*b*) the increasing financial complications and problems that are thrust upon it by unplanned and misguided or wasteful developments of the past and present.

As a consequence of the growing population and this understanding of urban complications, the duly constituted authorities are preparing compre-

* City Planner, Cambridge, Mass.

† Received by the Secretary, January 2, 1930.

hensive plans to guide and control the various improvement activities of the city for the general welfare. These may be considered under three subdivisions, as follows:

(1) Thoroughfare plans which after careful study are prepared to designate the thoroughfare system and routes for the various classes of traffic; the most appropriate and economical widths of streets and pavements; the ways and means of effecting grade separations at conflicting traffic intersections; and related problems.

(2) Zone plans which divide the city into districts or groups of districts of like land use, so as to provide better for the general welfare and orderly growth of business, industry, and residence, and to secure permanence in the investment in land and buildings, and promote general welfare and public safety.

(3) Plans for parks and open spaces to secure the much needed recreational facilities for the present generation and provide for the future needs, as well as the acquisition and development of land to preserve features of natural scenic beauty and historic interest.

It is through these plans and their co-ordination as comprehensive planning for the future that provision should be made for the location of an airport, and no airport site should be selected without reference to these important controlling factors.

Influence of the Thoroughfare System on the Airport Site.—The outstanding contribution of aviation to modern transport is that of conservation of time, which Mr. Baker has rated* as about 66⅔% saving for distances of 500 miles, or more, with a reduced advantage for shorter hauls. It is highly important, therefore, that the tremendous advantage in time saving be not diminished or lost through inadequate facilities on the ground between the airport and the points throughout the city or region from which air traffic will originate. It is the function of the thoroughfare plan to provide the best means of getting quickly and safely from one point in the community to another by creating direct routes, easy grades, and adequate width of pavement; by eliminating obstacles, such as railroad-grade crossings and highly congested street intersections; by the orderly control of traffic flow on highways; and by the provision of equally well-organized transit facilities.

Recognition of the Principle of Distribution of Land Uses, or Zoning in Relation to Airport Sites.—An early recognition that airport location is one of the established land uses, such as business, industry, or residence, and that it is not an isolated item, will aid materially in locating the airport on a site where it may become a permanent asset to the community. To locate a use outside its proper zoning district makes of it a non-conforming use, injurious to the welfare of its neighbors and subject to many restrictions on expansion and development. The general terminal airport is an activity displaying characteristics which, in their preponderance, will eventually classify it as an industrial activity. It must provide storage in quite large proportions; it transacts business during the day and more and more in the future will do business at night; it has adjacent manufacturing, or perhaps will have

* *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2314.

manufacturing, facilities for ships or parts; it must provide repair facilities; and, in its fundamentals, it must combine the facilities of business and industrial uses. Specialized airports, of course, may be appropriate for residential districts, where the chief function is the embarking or debarking of passengers. In general, however, the airport is an industrial establishment and should seek its location in an industrial district. The related question of airways is important, and some of the principles involved in the selection of an airport should govern in the location of the airway over a community. As pointed out in the paper,* it is important that the airport be so located as to reduce the actual flying over built-up sections and especially the residential areas.

Relationship of Recreational Facilities to Airports.—By appropriate design and co-ordination between the various agencies of the community government, the airport can be made a part of the recreational system of a community without in any way impairing its technical or operating efficiency.

The fundamental element in an airport is the landing field, about which are grouped other elements required for the complete functioning as a transportation agency. This landing field is an open space contrasting quite strikingly with the adjacent appendages. The airport in this open-space respect again contrasts with the built-up areas of the community, which, of course, vary in degree of density. It is, in part, this open area as one of the breathing spaces which makes it desirable to co-ordinate with the recreational system.

In addition to the benefit of the open space to the community, wherein there is free movement of air, there is always the value of the whole airport as an out-of-doors place of stimulating interest and activity, where imagination may be stirred and people aroused from routine ways by seeing and being associated with this modern system of transportation.

By skillful design the airport may be made a place of pleasure and amusement as a third form of recreational activity. Already one of the active pursuits in most airports is the sight-seeing trip which gives the visitor an opportunity to make his first flight or to cultivate a form of recreation for which a liking has already been established. Additional facilities for recreation would be established by the introduction of suitable restaurants, hotels, and allied places of amusement or sports.

Air transportation is making its way, and the city which does not adopt and plan measures for the airport will find itself in a few years left far behind in the relentless march of modern achievements.

S. N. Grimm,† Assoc. M. Am. Soc. C. E. (by letter).‡—The spectacular developments in design of large airships, both heavier and lighter than air, bring, as a result, a pronounced defferentiation of air traffic. While the large airships tend to serve the same purposes as ground and sea common carriers, the smaller aircraft confine themselves more and more to functions similar to those of the automobile and motor-boat.

* *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2313.
† Syracuse, N. Y.
‡ Received by the Secretary, January 2, 1930.

As time goes on great planes and dirigibles will be used for long-distance transportation along the principal routes of travel, with a limited number of stops, while the smaller planes will mostly fly comparatively short distances between the airports of a section of the country.

Many landing fields will serve exclusively the purposes of small aircraft, forming air highways, as it were. According to the data of the U. S. Department of Commerce, on July 1, 1929, there were 305 approved types of planes, 267, or 87%, of them having capacities of from one to six places. It is impossible to predict what will happen to this ratio in the future, but it is safe to assume that small aircraft will increase more rapidly in number than large airships. It is also evident that the future of private flying depends upon the improvement in design and in manufacturing of smaller planes, affecting their efficiency and cost, as much as upon the growth of the number of landing fields convenient for the purposes of private flying.

The problem of locating such fields to be used by private planes for landing, storing, and minor repairs only, may be solved in the manner suggested by the author* for major airports. However, the factors should be given different weights, according to the special purpose of the fields, the following being an example rather than suggestion:

Factors.	Weight.
(*a*) Topography	10
(*b*) Physical features	12
(*c*) Meteorology	12
(*d*) Location with respect to community served	5
(*e*) Transportation facilities	11
(*f*) Accessibility	25
(*g*) Legislative control	2
(*h*) Utilities available near site	3
(*i*) Cost	10
(*j*) Miscellaneous	10
Total	100

The weight of 25 points is given to the factor of accessibility of secondary airports, while the author gives to the same factor only 11 points for major airports. The reason for such a difference lies in the fact that, while the heavy air transport gains in its efficiency over ground common carriers in very long flights, the lighter air transport has to find its justification in an efficiency superior to that of highway travel in comparatively short distances.

Fig. 3, which is plotted in a manner similar to Fig. 2,† shows a marked decrease in rates of time saved through the use of an airplane, instead of an automobile, due to remote location of airports, for distances up to 360 miles. The following assumptions are made in this connection:

Airplane speed, in miles per hour	80
Automobile speed, in miles per hour	33
Time allowed, in minutes, for getting a plane ready and placing it in the hangar	15

* *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2316.
† *Loc. cit.*, p. 2314.

Time allowed, in minutes, for stop-over of plane and automobile after 200 miles	30
Time allowed, in hours, for stop-over of plane and automobile after more than 200 miles	10
Limit of distance traveled, in miles per day, by automobile	360

The author is certainly right in his assertion* that owners of private planes would not care to spend an hour traveling to the airport for a flight of like duration.

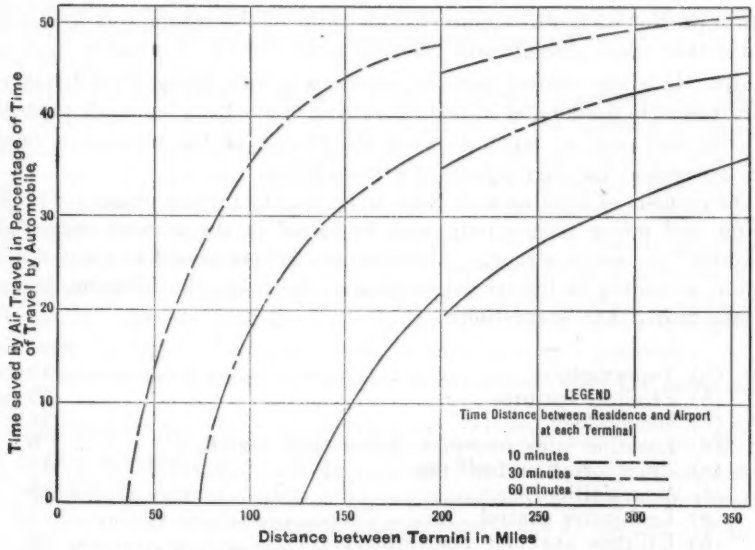


FIG. 3.—SAVING IN TIME ACCOMPLISHED THROUGH USE OF PRIVATE AIRPLANE INSTEAD OF AUTOMOBILE.

R. U. ST. JOHN,* ASSOC. M. AM. SOC. C. E. (by letter).†—The importance of applying sound engineering practice to the location of airports cannot be emphasized too strongly at the present time, when so many cities are contemplating such development.

The author presents very clearly the essential steps that should be followed in safe and adequate airport location. If a planner chooses a site in a haphazard manner without giving due consideration to the factors entering into the problem, he may cause large sums to be spent in improving a location, only to find that air traffic is afraid to use the field because of hazards, such as fog, smoke, or dangerous air currents. These hazards take on added importance where regular night-flying schedules are maintained.

The requirement for safe and unobstructed approaches to the field usually means that the location must be at some distance from the business center. However, every attempt should be made to secure a site which can be reached in a reasonably short time from the business district.

* Dist. Engr., Standard Oil Co.; Chairman, Eng. Comm., Airport Comm., Sacramento, Calif.

† Received by the Secretary, January 22, 1930.

While it is true that the saving in time, computed as a percentage over rail time, will be most imposing for trips of more than 500 miles, there is, nevertheless, a distinct saving to the business man on shorter trips, provided the airports are not too far from the business center.

Consider even as short a trip as 100 miles. If the airports at either end are 15 min. from the business district it is possible for a man to leave his desk at 10:00 A. M., spend two hours in the neighboring city, and be back to sign his mail at 4:00 P. M. Air travel, under favorable conditions, is more restful than any other means of transportation and a man would return from such a trip without feeling fatigued. To make the same trip by rail or automobile would take the business man away from his desk at least eight hours and he would return tired and travel-worn. The saving of travel time, computed as a percentage, may not be great, but the saving of business hours is of real value.

The study of meteorological conditions is of extreme importance in airport location. No one would think of building dock facilities for large boats drawing 30 ft. of water without first determining, by soundings, that the harbor would handle craft of that draft. Many communities, on the other hand, proceed with airport development, at considerable expense, without adequate data on wind direction and velocity.

The two most important meteorological factors from the aviator's standpoint are fog and wind direction and velocity. There is a real need for simple and inexpensive instruments that will record these data. If such instruments were available they could be installed for a period of six months or a year at proposed locations and would thus furnish the information accurately. They would be particularly valuable to smaller communities that do not have Weather Bureau Stations.

So far as the writer knows, there is no way of recording the actual presence of fog accurately other than by visual observation. In a recent study, the Airport Commission of the City of Sacramento, Calif., made use of recording hygro-thermographs in order to obtain data on fog conditions. These instruments were set up at representative stations throughout the area being studied. They furnished a continuous record of humidity and temperature during the three months' period when fogs occur in the region of Sacramento. By using the percentage of time during which 100% humidity was recorded, it was possible to rate the stations in the order of their fog-forming tendency. These results seemed to check fairly well with simultaneous visual observations; in other words, the stations where 100% humidity occurred for the greater proportion of time, were those at which the fog was most frequent and most persistent. It would be interesting to know whether similar results would be obtained in another region where the topography is uneven and where the fog belts are more sharply defined.

There are instruments for recording wind direction and velocity, but they are expensive to install and require considerable attention to maintain. These instruments record all air movements from a zephyr to a hurricane. For airport work the direction of light winds up to about 10 miles per hour is of

little importance, because ships can land "cross wind" under such conditions without hazard; but the direction and velocity of the stronger winds are important factors. There would seem to be no serious mechanical difficulty in turning out an inexpensive instrument that would record the higher wind velocities and show their direction within 10°, which is sufficiently accurate for locating runways.

The factors used as a basis of comparison of different sites should be definite and specific so that their value can be obtained by computation, as far as possible. The choice of a municipal airport location is a problem that usually interests real estate men with property to sell. If the factors used in determining the comparative merit of the different sites are assigned values on the basis of judgment alone there is opportunity for endless discussion and a danger perhaps of setting aside the recommendation of the body making the report. On the other hand, if the majority of rating factors are computed there is little room for argument.

As an example of a set of rating factors, most of which can be computed, Table 1 is taken from the report of the Airport Commission of the City of Sacramento.

TABLE 1.—Relative Weight of Factors Involved in Choosing the Location of an Airport at Sacramento, Calif.

Item No. (1)	Subject. (2)	Rating factors, ideal site. (3)	Rating factors, site selected. (4)
1	Freedom from dense river fogs	15	12.3
2	Freedom from bad air currents	10	8.0
3	Area of field	10	9.2
4	Configuration of field	5	5.0
5	Approaches, satisfactory areas under take-off	8	8.0
6	Favorable prevailing winds relative to runways	5	3.0
7	Prospective neighborhood development	4	3.0
8	Possibility of expansion	4	4.0
9	Location relative to axis of air travel	10	6.0
10	Distance from post-office	10	5.0
11	Distance from population center	10	6.3
12	Distance from railroad	4	2.0
13	Distance from airplane factory sites	5	2.0
	Total	100	73.8

This Commission spent nearly a year studying twelve proposed sites on a basis similar to that which the author of this paper advocates and was successful in securing the adoption of its report. The Sacramento Municipal Airport is now being constructed on the site recommended.

After assigning factors for the ideal site, the next step was to assign the proper values to each rating item for the various sites under consideration. The total of these values for any field would be its rating factor (see Column (4), Table 1). The assignment of the values was made by computation as far as possible, although a few of the items were necessarily a matter of judgment only.

1.—Freedom from Dense River Fogs.—The meteorological survey gave the information from which values of Item No. 1 were obtained. The four meteorological stations were rated in the order of their freedom from fog conditions, and this information was applied to the sites in the region adjacent to each station.

2.—Freedom from Bad Air Currents.—Values of Item No. 2 were matters of judgment and were assigned after numerous flights over the sites to determine the presence of any unusual air conditions. The proximity of trees or buildings causes uneven air currents and this fact was also used in determining this factor.

3.—Area of Field.—It was decided that an area of 250 acres represents the ideal site because it would give all the space needed at present and allow for a moderate expansion program. Sites larger or smaller than this ideal size were penalized in proportion. For example, the rating factor for a 400-acre tract would be $10 \times \frac{250}{400} = 6.25$.

4.—Configuration of Field.—The factor assigned to Item No. 4 was largely a matter of judgment. A rectangular field at least 2 500 ft. wide, with its length in the direction of the prevailing winds, would be the ideal shape. Odd-shaped fields on which it was difficult to lay out proper runways were penalized.

5.—Approaches, Satisfactory Areas Under Take-Off.—The total length of runways in the north and south and northwest and southeast directions was used as the basis for rating Item No. 5. If these lengths totaled 8 000 ft., the site received a full rating of 5. Shorter lengths were reduced in proportion.

7.—Prospective Neighborhood Development.—Factors for Item No. 7 were matters of judgment. If the trend was toward commercial or residential building, a low rating was given as this type of development close to an airport is hazardous. If the trend was only agricultural development, a high rating was given.

8.—Possibility of Expansion.—If all four sides of a tract were open to expansion, the full rating of 4 was given. If one side was blocked by a railroad or similar obstacle, the rating would be 3, etc.

9.—Location Relative to Axis of Air Travel.—Item No. 9 was appraised largely on the basis of judgment and conjecture as to future lines of air travel. In general, sites to the east of the town received the higher ratings because they are nearer the San Francisco-Reno Air Line.

10.—Distance from Post Office.—For any field, the rating for Item No. 10 was determined by the formula:

$$P = 10 - d \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which, P is the rating factor, and d is the distance, in miles, to the nearest post office. Ten was used as a limiting factor because it was felt that a site more than ten miles from the Post Office would have no value as an air mail field when so many closer sites are available.

11.—Distance from Population Center.—The value for Item No. 11 was computed the same as that for Item No. 10.

12.—Distance from Railroad.—Sites more than a mile from a railroad received zero rating. Those less than a mile were rated in proportion, the one having full rail connections adjoining the property receiving a full rating of 4.

13.—Distance from Airplane Factory Sites.—In general, Item No. 13 is affected by the same factors as Item No. 12, but special recognition was given to sites that had particularly good factory locations adjoining.

The data finally to be considered by the Commission consisted of the estimated cost of improving each one of the twelve proposed sites and a rating factor that indicated the value of the site as an airport. The last step was to divide the total estimated cost for each site by its rating factor to get a unit rating cost. This indicates the site which has the lowest cost per unit of rating and is a valuable indication of the best site from the standpoint of both rating value and cost.

It is hoped that one result of this paper will be to establish the general principles of airport location and to pave the way for a uniform rating schedule that will facilitate the comparison of proposed sites as well as completed airports.

JOHN F. PARTRIDGE,* M. AM. SOC. C. E. (by letter).†—The writer wishes to offer two suggestions which might be overlooked amid the many factors that will confront location engineers on airport developments.

The location of the airport for the small town should be made in the interests of the through transportation of mail, express, or passengers. The volume of business having the small town as its terminus is generally small by comparison, and time saving between the larger centers is the prime consideration. The smaller center will generously concede priority of convenience for the privilege of a tie to its larger neighbor. Transportation between the individual smaller centers on the route of flight is a result of convenience generally due to a scarcity of other available means of transport, and airport location should be again made in the interests of through service, either present, or possibly in the future. Utter disregard, by through service, of the demands of the smaller communities, however, is not in the best interests of air-transportation. The cultivation of public interest in aviation in the smaller town will result in increased air travel with its inevitable concentration around the larger communities.

Airport location in mountainous country should include consideration of the angle of slope from the tops of the neighboring mountains into and from the landing field. For smaller towns where through-transport planes stop for refueling, or the picking up of mail or passengers, it should be remembered that circuitous climbing to gain the necessary elevation to cross the mountains, may consume a considerable part of the total time between terminals. For larger centers, while this question of time lost in gaining altitude is still an important consideration, the other factors usually peculiar to a proper location of the airport may have to take precedence.

* Emprezas Electricas Brasileiras, S. A., Rio de Janeiro, Brazil.

† Received by the Secretary, February 5, 1930.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

HIGH DAMS
A SYMPOSIUM
Discussion*

BY MESSRS. B. F. JAKOBSEN, ALLEN HAZEN, and H. DE B. PARSONS

B. F. JAKOBSEN,† M. AM. SOC. C. E. (by letter).‡—In the paper entitled "Past Experience with High Dams and Outlook for the Future", A. J. Wiley, M. Am. Soc. C. E., defines§ arched gravity dams, as follows:

"In the case of the arched gravity dam there is a massive section capable of carrying the full load by its weight alone and, in addition, there is the resistance of the massive horizontal arch."

This is, in the writer's opinion, a correct definition and the only logical one. It is quite important that engineers should agree on a definition for gravity dams.

Elsewhere in his paper‖ Mr. Wiley lists the Exchequer Dam as an arched gravity type, although the writer has shown that this dam has a tension of about 40 lb. per sq. in. at the up-stream face, when concrete is assumed to weigh 152 lb. per cu. ft.¶ This dam, therefore, does not conform to Mr. Wiley's definition, since it will not stand by its weight alone, even when no uplift is assumed, and it should not be classed as a gravity dam. In discussing this subject further, Mr. Wiley states:‖

"In all these and in many others of similar design, the convergency of the radii is assumed to be offset by arch action and the uplift reduced, by pressure grouting and drainage, to an amount which can be taken care of by arch action."

This statement does not conform to the author's definition. That an arched gravity dam must be treated as such and not as a straight dam, the writer

* Discussion of the Symposium on High Dams continued from February, 1930, *Proceedings*.

† Cons. Engr. (La Rue & Jakobsen), Los Angeles, Calif.

‡ Received by the Secretary, November 13, 1929.

§ *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2321.

‖ *Loc. cit.*, p. 2319.

¶ *Transactions*, Am. Soc. C. E., Vol. 92 (1928), pp. 772 and 835; see, also, Mr. Wiley's discussion on p. 821.

has argued fully in a recent paper,* and he notes with satisfaction that D. C. Henny, M. Am. Soc. C. E., agrees with this and also includes uplift.† The writer can only insist that the factor of safety of a dam should be a calculable quantity and not something which is assumed; and he cannot agree to assume that arch action in general has certain beneficial qualities, unless these can be shown to exist and their magnitude calculated. Likewise, the writer is unwilling to assume that arch action in unknown amount can offset an unknown amount of uplift, as is done by Mr. Wiley. To assume no uplift, is to make the least conservative assumption possible and is not, in the writer's opinion, good engineering.

In discussing the arch section *vs.* the straight section, Mr. Wiley states:‡

"* * * if there are no open joints extending entirely through it, the slightest deflection of the dam as a gravity cantilever will cause an equal deflection in the horizontal arch and a division of the load in some proportion between the cantilever and the arch."

This assumption overlooks the fact that before shrinkage cracks appear, the concrete must be in tension; and, in that case, a deflection of the dam as a gravity cantilever will not cause the arch to take any part of the load. Before this can happen, the cantilever must be deflected sufficiently for the arch to be relieved of all tension produced by shrinkage of the concrete. It is this fact and others, which makes the calculation of the division of load between the cantilevers and the arches so problematical. It has convinced the writer and other engineers that gravity dams should be designed as such, irrespective of arch action, and that arch dams should be designed as such, irrespective of any possible cantilever action.§ This opinion was evidently shared by Mr. Wiley himself, who a few years ago wrote:‖

"It appears to the writer that in designing any arched dam of either full gravity, or reduced gravity, section all stresses should be considered as transmitted in either vertical or horizontal planes."

That arched gravity dams develop shrinkage cracks and leakage was brought out by the writer in discussing the Mulholland Dam¶ and E. W. Kramer, M. Am. Soc. C. E., recently stated:** "Some further reference to the Mulholland Dam will be made to prove that the contraction of concrete in a dam is real and not theoretical."

That dams not provided with contraction joints must develop cracks has been stated by Mr. Wiley,†† as follows:

"Nearly all high dams * * * are of necessity provided with construction joints if not contraction joints. If they are not so provided, they are

* *Transactions*, Am. Soc. C. E., Vol. 92 (1928), p. 766.

† *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2333.

‡ *Loc. cit.*, p. 2321.

§ "Stresses in Thick Arches of Dams," by B. F. Jakobsen, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 516, Article XXI; see, also, the writer's remarks, *Transactions*, Am. Soc. C. E., Vol. 92 (1928), p. 844.

‖ *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 116.

¶ *Loc. cit.*, Vol. 90 (June, 1927), p. 478.

** *Loc. cit.*, Vol. 92 (1928), p. 813.

†† *Loc. cit.*, Vol. LXXXIV (1921), p. 116.

sure to crack into sections at points which cannot be determined in advance. It is impossible to build, or at least to maintain, a large dam as a monolith."

The writer agrees with this statement, but does not see how Mr. Wiley's Conclusion No. 3* can be made to agree with it.

In discussing uplift inside the dams† the author appears to confuse external forces with internal stresses, and his attempt to reason uplift out of court does not carry conviction. Before it can be safely disregarded, it must be shown not only that it does not occur, but that it cannot occur in actual practice.

Under the heading, "Pressure Grouting of Contraction Joints," Mr. Wiley gives it as his opinion,* that such grouting would be of no appreciable value and that the joints would only be filled for a distance of about 15 to 20 ft. from the faces of the dam. In making this statement, he probably overlooked the elastic property of concrete, since he mentions filling the contraction joints. Some years ago, L. R. Jorgensen, M. Am. Soc. C. E., suggested‡ pressure grouting of the contraction joints of single arch dams with a view of compensating for shrinkage. By pressure grouting Mr. Jorgensen has been able to force the crest of a dam up stream, thereby demonstrating that the process was something more than merely filling the contraction joints. Fredrik Vogt, Assoc. M. Am. Soc. C. E., has applied for a patent on a multiple grouting system,§ which aims to equalize the arch stresses, as far as possible.

Mr. Wiley's Conclusion No. 4,* reads:

"If the contraction joints should open so as to prevent arch action, water entering the joints would reduce, by its back pressure, the effective head on the dam and compensate for the loss of arch action."

The writer does not understand this. As already stated, there may be no crack, but there may be tension, which would just as effectively prevent arch action as an actual crack. Possibly Mr. Wiley could work out an example in figures to illustrate and amplify his remarks.

Mr. Henny refers‖ to the experience with the Crystal Springs Dam in the San Francisco earthquake and concludes that very severe earthquake shocks may be borne safely by straight gravity dams. Professor F. L. Ransome refers¶ to the same dam, and neither calls attention to the pertinent fact that the dam referred to (the San Mateo Dam) has a section about twice that of the ordinary so-called gravity dam, which is designed with no allowance for uplift and is barely stable when the reservoir is full. The San Mateo Dam is 146 ft. high, 41 ft. wide at the crest, and 176 ft. at the base. The St. Francis Dam, which was a typical so-called gravity dam, had a crest width of 16 ft., and, for a height of 146 ft., its base width was 101 ft.** The San

* *Proceedings*, Am. Soc. C. E., November, 1929, papers and Discussions, p. 2325.

† *Loc. cit.*, p. 2323.

‡ "Improving Arch Action in Arch Dams," *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 316.

§ "Distribution of Major Stresses in Arch Dams Should Be Uniform Throughout the Structure," *Hydraulic Engineering*, December, 1929, p. 734.

‖ *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2333.

¶ *Loc. cit.*, p. 2345.

** *Loc. cit.*, January, 1929, Papers and Discussions, p. 223.

Mateo Dam has a stability of approximately four times that of most so-called gravity dams. It is evident that any experience with the San Mateo Dam does not apply to gravity dams in general.

Mr. Henny* calls attention to the fact that concrete shrinks in setting, largely due to the cooling action. Transversal shrinkage cracks in dams have been often observed, and Mr. Henny suggests the possibility of shrinkage cracks inside the dam, the planes of which would be more or less parallel to the up-stream and down-stream faces. Such cracks have appeared in a number of multiple-arch buttresses, and the possibility of similar cracks in gravity dams was first suggested to the writer by Dr. Vogt, in 1927, after he had inspected some high gravity dams. Some such cracks were found in a gallery of the Elephant Butte Dam, but these extended only a few inches from the surface. They were, moreover, very irregular, thus suggesting that they were only surface cracks. The writer thinks there is much less danger of such cracks developing in a gravity dam than in multiple-arch buttresses, but Dr. Vogt's suggestion merits careful consideration. So far as the writer knows, no such cracks have been found in gravity dams, and Mr. Henny does not mention any examples from his own experience. He refers to some contraction joints parallel to the axis of the dam as being provided in the design of the San Gabriel Dam. It is the writer's understanding that these joints have been abandoned, and in his opinion such an innovation should not be tried in the highest dam thus far attempted.

The one serious objection to proceeding with higher dams of the gravity type has not been mentioned by either Mr. Wiley or Mr. Henny. This is the question as to the accuracy of the fundamental assumption that the vertical stress distribution across a horizontal plane is linear. There is neither experimental nor theoretical verification of this assumption, which has been borrowed, somewhat uncritically, from the theory of bending of long beams. Whatever evidence of a theoretical and experimental nature there is, it is against the usual assumption. The tests of Messrs. Ottley and Brightmore in 1908, referred to by Mr. Henny,† as well as the tests by Messrs. Wilson and Gore, clearly indicate a non-linear stress distribution, as do also recent tests made by Dr. Vogt. Application of the principle of least work to this problem also gives a curved stress distribution and shows tension in the up-stream face, in cases where the ordinary theory gives compression. Even a small amount of tension in the up-stream face is extremely dangerous, in that there is a tendency for this to extend itself, especially if aided by uplift. The writer concludes, therefore, that tests to determine the actual stress distribution in gravity dams are badly needed before the height limit is materially increased.

As to whether a high arch dam is to be preferred to an arched gravity dam of equal height, that, in the writer's opinion, depends upon the site and its general characteristics, and no general rule can be laid down. If there is any question about the foundation being sufficient for a high arch dam, the writer would not consider a gravity dam either, unless it was of extraordinary

* *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2333.
† *Loc. cit.*, p. 2335.

dimensions, such as the San Mateo Dam. As between a properly designed arch dam and a gravity dam designed as outlined by Mr. Wiley, that is, on the assumption that the convergence of the radii and the influence of uplift are offset by arch action, the writer would invariably prefer the arch dam.

Professor Ransome presents* the viewpoint of the geologist. Any careful engineer will always insist upon obtaining competent geological advice and will then make his own investigations and tests and draw his own conclusions, bearing in mind that the responsibility is his. Geology is, relatively speaking, a general science. It is often limited to reasoning on possibilities and probabilities and from effect to cause, which is always a precarious and dangerous proceeding, unless its limitations are well understood. It is the geologist's office to inform the engineer of the general nature of the surroundings, the character of the rock, the probability of earthquake movements and shocks, their probable intensity and direction, etc. It is not the office of the geologist to express himself regarding the stability of a dam. That is a matter to be decided by the engineer after carefully made calculations based upon such experience as he may have at his disposal.

The geologist reporting on dam sites needs some general understanding of what kind of information the engineer wants, and the engineer should bear in mind that geology is general rather than specific. In this way the best co-operation will result between engineers and geologists, and such co-operation is absolutely necessary in order to avoid costly and fatal mistakes.

ALLEN HAZEN,† M. AM. SOC. C. E. (by letter).‡—At a time when more and higher dams are being built and when examples of dams that have failed or partly failed, are accumulating at an unprecedented rate, Mr. Henny's words§ of caution are most timely. Dams certainly do not follow the law of models and proportions that stand at one height cannot be taken as an argument for safety in a much higher dam of the same proportions.

In presenting the subject of safety in dam construction to the World's Engineering Congress in Tokyo, Japan, in November, 1929, the writer pointed out that the function of a dam is to absorb the water pressure upon it and to carry it to the sides and bottom of the valley and to distribute that pressure, together with that resulting from the weight of the dam, over and through enough of the natural material in place so as not to stress any part of it unduly. The pressures that must be absorbed and transmitted are among the largest that engineering structures of any kind are called upon to carry, extending to hundreds of thousands and even to millions of tons.

A strong, light dam may concentrate the pressure on the sides and bottom of the valley beyond the ability of the foundation material to withstand; and it avails nothing to have the dam strong enough if the natural material is not fully competent to hold the resulting stresses.

Mr. Henny does not mention the thought that has been held by some engineers for many years, namely, that a gravity masonry dam of dimensions

* *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2337.
† Cons. Engr. (Hazen, Everett & Pirnie), New York, N. Y.
‡ Received by the Secretary, January 14, 1930.
§ *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2327.

such that the line of thrust comes at the limit of the middle third, in reality, does not have any proper factor of safety. Many such dams have been built and some of them have failed. It may be true that those that stand depend upon tensile strength in parts of the masonry and resistance to torsion and shear and similar secondary means of resistance for any factor of safety that they may have. Furthermore, it may be that the weight of such a dam is in almost exact balance with the water pressure and is stressed by that water pressure so that if secondary sources of strength are excluded it is stressed almost to the point of rupture.

Nearly all dams in the last analysis are held by friction. It may be friction of masonry on masonry, or masonry on rock, or rock on rock. The friction in all these cases and at all possible lines of slipping must be sufficient to take up the thrust of the water pressure.

There is reason to think that the friction of masonry on some rocks and of the rocks on themselves may be much less than has been commonly supposed—and much less than would be prudently assumed for granite and other hard rocks. There is also reason to think that the frictional resistance of some rocks does not increase in proportion to pressure as is usually assumed.

The sliding of great dams presents many interesting problems which seem to have much in common with such phenomena as the slides at the Panama Canal and other great landslides. There is much uncertainty as to many factors, and increased weight is, in general, the surest road to safety.

H. DE B. PARSONS,* M. AM. SOC. C. E. (by letter).†—While the four papers, forming the symposium‡, contain many valuable suggestions, there are some statements that the writer would like to discuss.

Mr. Wiley§ lays stress on "the necessity of hard and durable rock foundations" as a rigid limiting condition for masonry dams. Such a condition certainly is much to be desired, but is not always available. A safe masonry dam can be built on "poorer" rock to heights that could be considered high, if given a suitable section. For a thin arch dam, however, "good" rock is essential. It is questionable whether the horizontal arch action in an arched gravity dam, having the same cross-section as a straight gravity dam, would offset the reduction in gravity action due to the convergency of the radii. The amount of horizontal arch action obtained would depend on the length of the radius and on the height of the dam. The water pressure on a gravity dam is considered as carried by the cantilever action. Assuming the base rigidly fixed to the rock, the deformation of the masonry would be a function of the height. If the same dam were arched, the radius would have to be relatively short or the dam very high before the compression of the masonry, due to deformation, would cause an horizontal arch thrust that would be truly effective at the buttress ends. With a long radius, the deformation probably would be taken up by closing vertical contraction joints.

* Prof. Emeritus, Rensselaer Polytechnic Inst.; Cons. Engr., New York, N. Y.
† Received by the Secretary, January 21, 1930.
‡ *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2317.
§ *Loc. cit.*, p. 2319.

A reading of Mr. Wiley's paper leaves the impression that hydrostatic uplift is not an important element, especially under arched gravity dams. All reported observations under actual dams show the presence of uplift.* Uplift would seem to be an actual force, like gravity or reservoir water pressure. Recorded observations show that grouting under the up-stream edge of the base does not eliminate uplift, and the assumption that it will is not founded on fact. It may lessen the uplift, but the reduction is questionable. When grouting is practiced, it would seem safer to use low pressures. The high pressures (say, from 80 to 150 lb. per sq. in.) often used may open the seams in a rock foundation and thereby cause increased uplift.

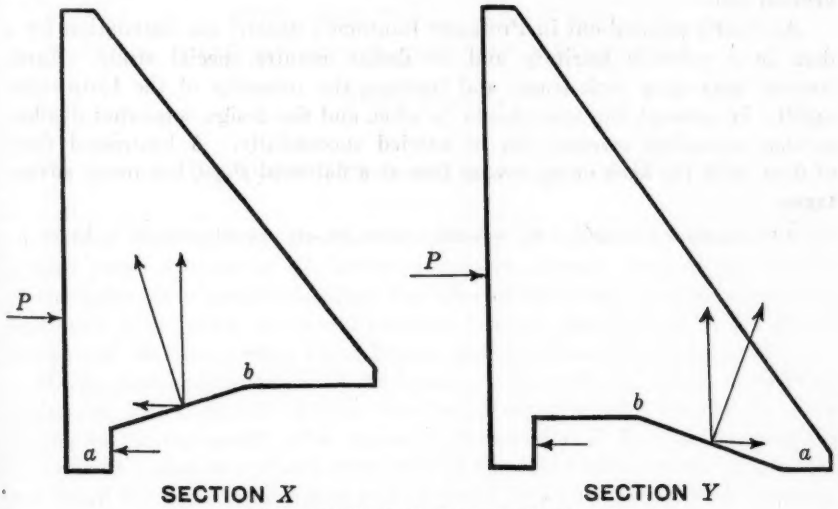


FIG. 7.—EFFECT OF FORM OF BASE OF DAM ON SLIDING.

The conclusion† may be true that "there is no uplift * * * in foundation rock which is free from seams, nor in a perfect connection between the dam and the bed-rock", but such perfect conditions are not found at the sites for high dams. It would not be safe to design an important dam or a high one under such assumptions. Uplift under the base of a dam is a natural force, and the shape or material of a structure does not change this law of Nature. If it exists under a straight structure, it would also exist under an arched one.

In Mr. Henny's paper hydrostatic uplift under dams is given recognition.‡ The writer believes that it cannot be entirely "prevented" although it may be reduced. The form of the dam base may have an effect on sliding due to hydrostatic pressure. In Fig. 7, two sections for gravity dams are shown that differ in their base planes. Hydrostatic head acts normally to the surface. With Section *X*, the head acting against the cut-off, *a*, and the hori-

* "Hydrostatic Uplift Under Masonry Dams", by H. deB. Parsons, M. Am. Soc. C. E., *Paper No. 313*, World Eng. Congress, Tokyo, Japan, November, 1929.

† *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2326.

‡ *Loc. cit.*, p. 2334.

zontal component acting on the slope between *a* and *b*, would be opposed to the reservoir pressure, *P*, and, therefore, would diminish the tendency to slide. With Section *Y*, the horizontal component of the hydrostatic head acting on the slope between *a* and *b*, would be in harmony with the reservoir pressure, *P*, and would thereby increase the tendency to slide.

Deflection of the masonry, acting as a cantilever, affects the stresses at the up-stream edge of the base. At this edge, uplift is usually at a maximum. Therefore, the stress across the base joint, between the rock and the masonry (or concrete), may be changed from compression to tension, and the intimate contact between masonry and rock may be weakened or the rigid connection lost.

As clearly pointed out in Professor Ransome's paper,* the foundation for a dam in a volcanic territory and its design require special study. Earth tremors may open rock seams and increase the intensity of the hydrostatic uplift. In general, the base should be wide, and the design somewhat flexible, so that secondary stresses can be carried successfully. A buttressed form of dam, with the back or up-stream face at a flattened slope, has many advantages.

* *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2337.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

CONSTRUCTION OF THE JAMES RIVER BRIDGE PROJECT

Discussion*

By Wilson T. Ballard, M. Am. Soc. C. E.

Wilson T. Ballard,† M. Am. Soc. C. E.—Two points mentioned briefly in this paper are worthy of further discussion, namely, preliminary studies of the subsurface conditions along the sites of the three bridges, and actual load tests made upon reinforced concrete bearing piles driven in the trestle portions of the Nansemond River Bridge and the James River Bridge.

Subsurface explorations of the bridge sites were made by borings, by driving timber test piles, and by driving reinforced concrete piles similar in every respect to the permanent piles driven in the structure. The borings and the behavior of timber test piles during driving furnished information upon which was based the tentative decision as to general type and layout of the bridges, and the preliminary estimate of cost of the entire project. However, it did not furnish sufficiently reliable information upon which to base a final schedule of the lengths to be used in casting the concrete piles nor in determining the location of piles of various lengths in the bridges. For example, concrete pile lengths indicated by the behavior of timber piles driven in Chuckatuck Creek were from 15 to 20 ft. too long. As a further example, reinforced concrete piles could not be driven to more than one-half the desired penetration at the sites of the lift-span piers of the James River Bridge, whereas timber piles were driven successfully to the desired penetration of from 40 to 45 ft. through a hard blue marl. Had the material through which these piles were driven, been sand it would have been possible to sink reinforced concrete piles to the desired penetration by the use of a jet.

The driving of test piles of the same size and kind as designed for the finished structure furnished very reliable data for scheduling the pile

* This discussion (of the paper by R. C. Wilson, Esq., and Herbert B. Pope, Assoc. M. Am. Soc. C. E., presented at the meeting of the Construction Division, New York, N. Y., January 17, 1929, and published in November, 1929, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Vice-Pres. The J. E. Greiner Co., Baltimore, Md.

lengths to be cast for various locations. Therefore, such piles were spotted at various points ahead of the location where driving of permanent bearing piles was in progress, and the casting schedule was governed by information thus obtained. As a result, there remained only one unused pile after all the permanent bearing piles had been driven.

In view of the great cost of making subsurface exploration of bridge sites similar in length and character to those of the three bridges of the James River Project, it is believed to be wise economy to confine the investigation to the driving of test piles exactly similar in every respect to those intended for use in the permanent structure. The pile-driver used on the test piles should be of the same type and weight as that to be used in driving the permanent piles. The information thus obtained may be supplemented by sufficient test borings to determine the character and depth of the underlying soil strata.

The load tests made on test piles and permanent bearing piles brought out interesting facts which add confirmation to the belief that the bearing value of large concrete piles driven in various types of soft material cannot be determined by the pile-driving formulas in general use. As stated by the authors,* each test was made by mounting a steel tank upon the pile and filling it with water. The load was applied rapidly until it reached about 30 tons and thereafter more slowly. Observations of settlement were taken during and after loading. Of the eight tests made on permanent piles the time that elapsed during the application of the load was from 1½ to 3 days. The maximum test load was allowed to remain in place for 2 to 5 days, except in the case of two piles in the Nansemond River, from which it was removed after one day.

During the summer of 1927, several reinforced concrete test piles were driven along the site adopted for the James River Bridge, in that length of the river where borings and soundings indicated that greatest pile lengths would be required. This part of the river included a distance of about 6 000 ft. along the center line of the bridge and lay south of the present main channel. It may have been a deep channel in times past which was gradually deepened with the general subsidence of the eastern seaboard and filled up with clay and silt brought down by the river. These test piles varied in length from 90 to 114 ft. and were 24 in. square in cross-section. They were driven by a No. 1 Vulcan single-acting hammer, with a ram weighing 6 500 lb. and a 3-ft. stroke. None of them could be driven to refusal in the soft bottom. Four were subjected to test loads sufficient to produce failure and the results were compared with calculated bearing values found by the use of the following pile-driving formulas:

Greiner formula:

$$P = \frac{1.07\,w\,h}{S + 0.1} \qquad (1)$$

Engineering News-Record formula:

$$P = \frac{2\,w\,h}{S + 0.1} \qquad (2)$$

* *Proceedings*, Am. Soc. C. E., November, 1929, Papers and Discussions, p. 2365.

Wellington formula:

$$P = \frac{2\,w\,h}{S + 0.1\frac{W}{w}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

wherein:

P = allowable design lead on pile, in pounds.
h = stroke of hammer, in feet.
S = penetration of pile, in inches, under final blow.
w = weight of striking parts of hammer.
W = weight of pile, in pounds.

Table 1 shows the results of the test compared with the calculated bearing values of the piles found by the use of Equations (1), (2), and (3).

TABLE 1.—Comparison of Results of Actual Load Tests with Bearing Values Computed by the Use of Formulas.

Pile No.	Equation.	Penetration, S, in inches, under final blow.	Computed bearing value, P, in tons.	Maximum test load that caused no settlement, in tons.	* Load that produced failure, in tons.	Factor of safety based upon bearing value, computed by equations.
1	(1)	0.5	17.4	20	85	4.9
	(2)		32.5	20	85	2.6
	(3)		14.7	20	85	5.8
2	(1)	0.095	53.5	30	121	2.3
	(2)		100.0	30	121	1.2
	(3)		17.4	30	121	7.0
3	(1)	0.25	29.8	30	85	2.9
	(2)		55.7	30	85	1.5
	(3)		17.0	30	85	5.0
4	(1)	0.226	32.1	30	103	3.2
	(2)		62	30	103	1.7
	(3)		15.3	30	103	6.7

* The load that produced failure is defined as that which produced rapid settlement that continued until part or all of the test load was removed.

These results showed the futility of attempting to govern the driving of the permanent piles by formula. As a result of these preliminary tests, it was decided to attempt to drive the permanent bearing-piles to a final set of not more than ¼ in. per final blow under the No. 0 Vulcan hammer (weight of ram, 7 500 lb., and stroke of 3.5 ft.).

When the driving of the permanent piles was commenced in the Nansemond River in January, 1928, it was soon evident that this could not be done with piles of reasonable length. It was then decided to continue driving, to observe and record carefully the behavior of the piles, and to subject each of those that offered the least resistance to driving to an actual test load equal to twice the total load the pile would be required to carry in the finished bridge. Four of those driven in the Nansemond River were selected for test; two were 65 ft. long by 18 in. square, one, 70 ft. long by 18 in. square, and one, 95 ft. long by 24 in. square. Boring data obtained along the site of the bridge indicated that the three 18-in. piles penetrated from 10 to 15 ft. of soft mud and 30 to 35 ft. of soft blue clay or soft peat, while the 24 in. by 24-in. pile penetrated 15 ft. of soft black mud, 5 ft. of soft peat, 30 ft. of soft blue clay, 23 ft. of silty peat, and 15 ft. of hard, fine sand.

However, the driving record of the 95-ft. pile would indicate that it probably did not penetrate much, if any, sand as it drove with comparative ease until its tip was 88.7 ft. below mean low water. The number of blows required to produce the final foot of penetration of each of these four piles is shown in Table 2 (Piles *A*, *B*, *C*, and *D*), together with the results of the load tests.

TABLE 2.—RESULTS OF LOAD TESTS.

Pile.	Length, in feet.	Size, in inches square.	Number of blows for final foot.	Maximum design load, in tons.	Test load, in tons.	Total settlement under test load, in inches.
A	70	18	24	34.4	68.8	9/64
B	65	18	18	34.4	68.8	1/4
C	65	18	32	34.4	68.8	1/4
D	95	24	28	42.0	94.0	7/16
E	115	24	9	40	80	1/8
F	115	24	5	40	80	3/16
G	115	24	10	40	80	17/32
H	115	24	8	40	87.7	6

After piles in the bent that included Pile *A* had been in place for 24 hours, the driver was placed on a pile adjacent to Pile *A* and an attempt was made to drive it farther. Fifty blows of the hammer drove the pile ¾ in. After a brief interval, twelve more blows produced no further penetration.

After the removal of the test load, Pile *D* (Table 2) recovered 5/16 in., indicating that the permanent settlement caused by the load was ⅛ in. The three 18-in. piles each recovered approximately ⅛ in. on removal of the test load.

As a result of the tests of these four piles confidence was established in the fact that the load-carrying capacity of concrete piles that could be driven to a set of from ½ in. to ¾ in. through strata such as were encountered in the bottom of the Nansemond River, would be safe for the loads to be imposed by the bridge in service.

During the driving of piles for the James River Bridge, several could not be driven to a final set of ½ in. to ¾ in. with pile lengths of 115 ft. It was decided, therefore, to make load tests on four of these piles which offered the least resistance to driving under the No. 0 Vulcan hammer, as used in Nansemond River. The four selected were each 24 in. square and 115 ft. long. According to borings made along the bridge site, each of these piles penetrated about 30 ft. of water, 50 ft. of stiff blue clay, from 5 to 10 ft. of soft blue clay, and from 20 to 25 ft. of sand. The behavior of these four piles during driving, however, would indicate that it was very doubtful if they penetrated any sand, as the number of blows of the driver for the final foot of penetration did not exceed ten for any of them. As the bottoms of the Nansemond River and James River, particularly the latter, vary greatly in character within short distances measured along the center lines of the bridges, and since they have at frequent intervals deep pockets of soft material, it seems quite likely that such pockets may have occurred at the locations of the four piles in the

Nansemond and the four piles in the James River that were tested, without being indicated by the boring data.

Table 2 shows a summary of the driving data and load-test results of the four piles (*E*, *F*, *G*, and *H*) in the James River Bridge. The test on Pile *H* is considered unfair because a severe storm with high wind prevailed. Heavy waves beat against the test tank and this probably caused the pile to loosen and settle as noted; settlement stopped when the filled test tank came to rest on adjacent piles. It should be stated that during the first three days of the storm, the pile settled only 1½ in. under a test load of 53 tons.

As a result of these tests it was decided that piles of lengths equal to those tested, and which behaved in a similar manner, could be relied upon to carry safely the maximum design load of 40 tons per pile. The three bridges have been in service for more than a year and there is no apparent evidence of pile settlement in either of them.

It is felt that the foregoing facts show that, when it is intended to support structures on piles, it is wise economy to limit subsurface exploration of sites similar to those described herein, to the driving of test piles identical in every respect to those intended for use in the permanent structure. Consideration of the results of the load tests described, adds confirmation to the belief that, in pile-driving of the nature described, the various formulas in general use cannot be relied upon to govern the driving, nor to determine the economic safe carrying capacity of the piles. Load tests should be made for this purpose.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

GENERAL SPECIFICATIONS FOR STEEL RAILWAY BRIDGES

PREPARED BY COMMITTEES FROM
THE AMERICAN SOCIETY OF CIVIL ENGINEERS
AND
THE AMERICAN RAILWAY ENGINEERING ASSOCIATION

Discussion*

BY MESSRS. C. H. CHAPIN, HUNTER MCDONALD, A. F. BLAESS, AND C. R. HARDING

C. H. CHAPIN,† ESQ. (by letter).‡—The use of modern specifications on various subjects has led the writer to draw the following conclusions which apply to a certain extent to nearly all specifications:

(1) That the authors try to cover the subject thoroughly.

(2) That the authors are thoroughly conversant with the technical side of the subject.

(3) That the authors are executives or instructors who are now engaged in design only in a supervisory capacity and, consequently, fail to note the great amount of detail work made necessary by some of the methods they advocate.

(4) That the authors have before them the ideal or standard structure rather than the special one. It is in the latter case that specifications are most needed and in which they so often fail.

(5) That the wording of specifications important to the designer is often so vague as to permit of more than one interpretation.

Many times during the progress of the design of a recently completed, badly skewed, multiple-track viaduct, the writer was called upon to decide between two interpretations of the same specification, or to settle an argument arising over some point not completely covered by the specifications.

* Discussion of the General Specifications for Steel Railway Bridges, continued from March, 1930, *Proceedings*.

† Designing Engr., C. & O. Ry., Richmond, Va.

‡ Received by the Secretary, January 7, 1930.

He did so, not, however, by virtue of his superior knowledge of the subject, but merely because he had the authority to do so. That, though, is bad business and should not be necessary with modern specifications.

The comments that follow, which are the personal views of the writer and not necessarily those of the company by which he is employed, are prompted mainly by a desire to simplify specifications so that a safe, economical structure can be produced with the least amount of work and so calculated that two designers can get the same result.

*Article 102.**—Riveted trusses have many advantages over pin-connected trusses. The use of the latter should be discouraged rather than encouraged.

Article 203.†—There is very little, if anything, to be gained by the use of the simplified loading for spans of 40 ft., or more. Tables and influence lines make the use of the engine loading as easy as any other, and the engine should be used throughout.

What live load shall be used for solid floors having transverse beams? The live load for three or more tracks should be stated as a percentage of the total load rather than as the full load on two tracks and 90% of the load on all others.

Article 204.‡—The notation, "l = length in feet, of the span which is loaded to produce the live load stress", has been interpreted to mean the length of that part of the span actually loaded (which is seldom the full length of the span) and as the full length of the span. For deck and through girders and for truss chords, l should be either the total length of the track affecting the member, regardless of whether or not the load covers its entire length, or the length, center to center, of bearings of the girder or truss, preferably the latter. For truss members, l might be taken as the distance from the center of the panel toward the unloaded end adjacent to the point in question to the center of bearing at the loaded end.

Taking full impact on one track and none from the other will add greatly to the designer's work in the case of skewed spans, as it will either prevent the use of influence lines or make their use a complicated problem. The method proposed is evidently an approximation and undoubtedly another one could be made involving the use of a smaller impact coefficient and the total live load that would give practically the same result and permit the easy use of influence lines.

In stating that for girders and trusses of double-track bridges, the impact shall be only from the nearest track, the Committees evidently had in mind the longitudinal girders or trusses. How would the impact be applied to the floor-beams of double-track truss spans, or to the cross-girders of double-track viaducts, where both tracks are the same distance away and produce the same stress? Impact on only one track would produce an unsymmetrical loading but, since the loading could be reversed, the resulting girder would be symmetrical. Is it the intention to apply the impact in this way? What impact should be used for steel bridges having concrete and ballast decks? Does the

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2650.
† *Loc. cit.*, p. 2651.
‡ *Loc. cit.*, p. 2652.

impact for bridges designed for electric traction take into account electric locomotives having connecting rods and counterweighted drivers?

*Article 205.**—Is centrifugal force to be taken from one track or from as many tracks as produce stress in the member? If the latter, is impact to be taken on all tracks?

Article 210.†—Could not the longitudinal force be taken as a percentage of the total load to avoid having to separate the load from the drivers? The percentage could vary, if necessary, with the span length, and tables could be used for getting the load.

Two positions of the live load are required to obtain the maximum live stress in the four columns of a viaduct tower. How, then, should the live load be placed to give the column stress due to longitudinal force that should be combined with the maximum live column stress? Would not a uniform load be satisfactory for this purpose? In calculating the longitudinal force on a viaduct tower, should the load between expansion joints be considered as acting, or that between the mid-points of the spans adjacent to the tower? The specifications should contain answers to all these questions which cause countless arguments among designers.

Article 300.†—The work would be simplified by using two-thirds of the calculated dead load as the design dead load and the unit stresses given in Article 301, especially when Article 302‡ has to be considered.

Article 301.§—The allowable shear on the net section of plate-girder webs should also be given. Very often detailers, with nothing to guide or limit them, design connections that produce a shear on the net section of 15 000 lb. per sq. in., or more. Many changes in shop drawings could be avoided if the shear on the net section were limited by the specifications to not more than 12 500 lb. or 13 000 lb. per sq. in. Shear in rivets, pins, and bolts should be given.

The bearing on concrete of 600 lb. per sq. in. seems rather high. The concrete under girder and truss shoes is subjected to hard punishment, and, until shoes become so large that they are difficult to manufacture and handle and to accommodate on the masonry (which is not true of the present loading), it should be desirable to keep the bearing on concrete down to not more than 500 lb. per sq. in. This also would better provide for future loading.

Article 401.‡—For railroad bridges, all of which are exposed to the weather, $\frac{5}{16}$ in. should be the minimum thickness of material.

Article 407.‖—Longitudinal tower bracing should also be included. Very often long tension members must be made heavier than would be necessary to carry the stress in order to keep the slenderness ratio, $\frac{l}{r}$, down to the maximum, and there is nothing gained by carrying this excess strength into the connections.

---

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2652.
† *Loc. cit.*, p. 2653.
‡ *Loc. cit.*, p. 2655.
§ *Loc. cit.*, p. 2654.
‖ *Loc. cit.*, p. 2656.

*Article 428.**—The top flange of a deck girder is subject to marked corrosion because of the destruction of paint due to movement of ties, to brine drippings, etc., and scores of girders must be repaired annually. In the case of girders without cover-plates, it is necessary to replace the flange angles, or, if possible, reinforce them with cover-plates the holes for which must be drilled in the field. Either method is considerably more expensive than replacing an existing plate. All girders, at least all deck girders, should have a cover-plate the full length of the top flange.

*Article 430.**—Does the splice have to develop fully the web in shear or only the shear actually occurring at the point of splice?

*Article 431.**—When using the simplified loading for shear for spans of 40 ft. or more, what vertical load should be used in combination with the longitudinal shear to obtain the rivet pitch?

*Article 433.**—In Part (*b*), "the depth between flanges" has been interpreted to mean the clear distance between flange angles, the distance from center to center of gravity of the flanges, and the distance back to back of flange angles. This should be stated more clearly.

Article 439.†—In the third paragraph of this Article it is not clear how transverse and longitudinal bracing can transmit the longitudinal force to both sides of a tower. Horizontal bracing between diagonally opposite columns should be used in conjunction with the transverse and longitudinal bracing, and, if horizontal bending is to be kept out of the cross-girders, a horizontal traction truss framed between the columns and stringers would be required at each bent.

Tractive force and the stresses resulting therefrom cause more arguments and consequent loss of time than any other one thing connected with bridge design. Every bridge designer has his own pet way of calculating the stresses; no two methods are alike; all kinds of results are obtained and textbooks "fight shy" of it, but specifications require that it be considered. If the Committees in their studies prior to the writing of the specifications discovered any method they consider authentic, of calculating the tractive stresses in the various members of a viaduct tower (more particularly in one in which the bents are skewed with the tracks and the columns unsymmetrically placed with respect to the tracks), reference to this method should be made either in the specification proper, or in a footnote. Right or wrong, at least it would require every designer to use the same method and, incidentally, put an end to some of the writer's troubles.

Article 442.‡—This is assumed to mean $\frac{1}{2}$ in. either way.

Article 448.‡—The length of all members should be changed to produce the desired result instead of increasing the length of the top chord $\frac{1}{8}$ in. for each 10 ft. The rule-of-thumb-method seems out of place in a modern specification and is not in accordance with current practice.

These comments refer to only a few of the many Articles contained in the proposed specifications. In other respects the subject seems to have been

* *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2659.
† *Loc. cit.*, p. 2660.
‡ *Loc. cit.*, p. 2661.

covered very thoroughly, and it is to be hoped that a free discussion by all who may be interested will result in an air-tight, "fool-proof" standard specification that will apply to all cases.

HUNTER MCDONALD,* PAST-PRESIDENT, AM. SOC. C. E. (by letter).†—These specifications as a whole are a decided improvement over the present standard specifications in use by the American Railway Engineering Association. The two Committees are to be congratulated on the splendid work done toward finally reaching a specification for railway bridges which would be acceptable to interests concerned; but it seems to the writer that their work should be such as can readily be co-ordinated with a standard form of construction contract adapted to general use by railway engineers and those representing other clients.

For example, it should be susceptible of use in connection with the proposed "Standard Construction Contract" as submitted by the Special Committee of the Society in 1926,‡ provided it is the intention of the Societies that previously fostered this movement to continue their efforts until such a form of contract is agreed upon and adopted.

The transfers suggested herein are based on the assumption that the General Specifications for Steel Railway Bridges are to be used in connection with such a standard contract. Upon this assumption there should be transferred to the Standard Construction Contract such items in the General Specifications for Steel Railway Bridges as belong appropriately thereto.

In order to make such transfer it will be necessary to modify certain Articles in the present Committee report, somewhat as follows:

(*a*) Article 1§ of the Specifications is already covered by Article 1‖ of the "Standard General Conditions for Engineering Construction."

(*b*) Article 2§ properly belongs in the General Conditions and its contents are, to some extent, indicated under "General Notes", pertaining to Standard Construction Contracts.¶

(*c*) Articles 3 to 8,§ inclusive, can be more appropriately covered by "Articles" already included in the "General Conditions for Engineering Construction".**

(*d*) Likewise, Articles 543 to 547,†† inclusive, as well as Section 7‡‡ of the General Specifications, should be transferred to their appropriate Articles in the General Construction Contract.

These suggestions are made also on the assumption that the "Invitation for Proposals" (Article 2§§) will necessarily include or make reference to the Standard Construction Contract,‖‖ the Standard General Conditions for

* Chf. Engr., N. C. & St. L. Ry., Nashville, Tenn.
† Received by the Secretary, January 29, 1930.
‡ *Proceedings*, Am. Soc. C. E., March, 1926, Society Affairs, p. 223.
§ *Loc. cit.*, December, 1929, Papers and Discussions, p. 2649.
‖ *Loc. cit.*, March, 1926, Society Affairs, p. 232.
¶ *Loc. cit.*, p. 225.
** *Loc. cit.*, March, 1926, Society Affairs, p. 233.
†† *Loc. cit.*, December, 1929, Papers and Discussions, p. 2665.
‡‡ *Loc. cit.*, p. 2666.
§§ *Loc. cit.*, p. 2649.
‖‖ *Loc. cit.*, March, 1926, Society Affairs, p. 230.

Engineering Construction and the General Specifications for Steel Railway Bridges, and that it will be made a part of the contract.

Pending the adoption of a standard form of contract a number of these suggested transfers could be appropriately made, no matter what form of contract is used.

In his work the writer uses a set of general conditions adaptable for buildings and roadway construction in connection with the Standard Construction Contract as proposed by the Special Committee of this Society. In bridge work and in building work which involves steel construction, the practice is to make reference in the contract to the present standard specifications of the American Railway Engineering Association. In all such forms it is necessary to provide blank lines for special types of construction in which the standard forms in some instances do not apply.

A. F. Blaess,* Esq. (by letter).†—The Committees have made a radical change in the engine for which structures shall be designed. For years, engineers have been trying to substitute a wheel loading that would take the place of Cooper's equivalent loadings, but, while several have been suggested, none has met with favor. As far as the writer has been able to ascertain, none has come any closer to actual conditions than the Cooper loadings, which are so well established that, unless an engine can be devised that will unquestionably give better results, the writer thinks it better to retain the present system. It is very questionable whether a better standard engine can be developed as long as engines actually used by the different roads are of so many different types.

The Committees have also made a slight change in the impact formula from that adopted by the A. R. E. A. (See Article 204.‡) The new formula gives results that differ only about 8% from the old one. There is not sufficient reason to accept a formula that would be so little different from the present one.

There is also a rather decided change in the effect of impact with additional tracks on multiple-track bridges. This subject is purely a matter of opinion, and there is no new line of reasoning with which the writer is familiar that would justify making the change.

There are also some minor points throughout the specifications, but they will have no consequential effect, one way or the other.

C. R. Harding,§ M. Am. Soc. C. E. (by letter).‖—These specifications are representative of modern thought and present-day knowledge with respect to the design of steel railway bridges and the materials that enter into their construction. They combine the best features of other specifications heretofore used, and no effort has been spared to obtain a greater clarity and better arrangement of the entire subject.

---

* Chf. Engr., I. C. R. R. System, Chicago, Ill.

† Received by the Secretary, February 1, 1930.

‡ *Proceedings*, Am. Soc. C. E., December, 1929, Papers and Discussions, p. 2652.

§ Asst. to Pres., S. P. Co., San Francisco, Calif.

‖ Received by the Secretary, February 4, 1930.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

PLASTIC FLOW IN CONCRETE ARCHES

Discussion*

BY MESSRS. B. F. JAKOBSEN, AND C. A. P. TURNER

B. F. JAKOBSEN,† M. AM. SOC. C. E. (by letter).‡—The author presents a theory of bending in which the stress-strain relation of Equation (9)§, $\varepsilon = k\,\sigma^m$, is substituted for Hooke's law, for which $m = 1$. As presented by the author, the constants, k and m, are the same for tension as for compression.

The advantage of Hooke's law is its simplicity, as may be seen by comparing the ordinary formulas with those presented by the author; another advantage is that if it errs, as it does for high stresses, it errs on the side of safety. An analysis by the writer‖ of the test on the arch at Elevation 30 of the Stevenson Creek Dam shows that Cain's formulas, based on a constant modulus of elasticity, give stresses which agree well with the measured strains. In this case the stresses are fairly small and they are compressive throughout. An attempt to deal with the arch at Elevation 40 failed,¶ probably because the Cain formulas give considerable tension and either cracks were produced or a shifting of the neutral axis took place, and possibly both phenomena occurred.

It appears to be a moot question whether or not Hooke's law applies for stresses well below the breaking stress, although scientific men have been occupied with a study of this phenomena for two centuries. Bulffinger proposed the exponential law in 1729 and Hodgkinson considered it in 1822. The parabolic law and recently the hyperbolic law have also been considered.** Bach states†† that for compression of concrete, which has not been previously

* This discussion (of the paper by Lorenz G. Straub, Jun. Am. Soc. C. E., published in January, 1930, *Proceedings*, but not presented at any meeting of the Society) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr. (La Rue & Jakobsen), Los Angeles, Calif.

‡ Received by the Secretary, January 27, 1930.

§ *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 60.

‖ *Loc. cit.*, May, 1929, Papers and Discussions, p. 1241.

¶ *Loc. cit.*, p. 1247.

** Bach, "Elasticität und Festigkeit," Ninth Edition (1924), p. 109, footnote.

†† *Loc. cit.*, p. 84.

stressed, the deformations increase faster than the stresses. He gives various values for the coefficients, k and m. Stanton Walker, M. Am. Soc. C. E., comes to the same conclusion* for concrete in compression. The late J. B. Johnson, M. Am. Soc. C. E.,† showed that for tension Hooke's law applies in general until near the breaking strength and states‡ that the modulus of elasticity for stone concrete in tension is only 0.7 as great as in compression. Hool and Johnson give stress-strain curves for concrete in compression,§ which show that Hooke's law applies for stone concrete up to a stress of more than 2 000 lb. per sq. in. at the age of 1 year. Messrs. Taylor and Thompson state that for compression the deformation curve is approximately a parabola, but that the modulus of elasticity is nearly constant for about one-third the ultimate strength, that is for all normal working stresses.‖ The modulus for tension they state¶ is probably the same as for compressions, though but few tests have been made. Tests by Mörsch indicate that for concrete the deformations increase faster than the stresses, both for tension and for compression and that the modulus for tension is slightly larger than for compression.**

Tests have been carried out recently in the Research Laboratory of the Portland Cement Association covering the modulus of elasticity of concrete in both tension and compression, using the most accurate technique and apparatus. As a result of these tests F. R. McMillan, M. Am. Soc. C. E., has stated to the writer under date of January 11, 1930, that he does not accept the exponential law, as Hooke's law applies up to normal working stresses and generally considerably beyond these; also that the modulus in tension is substantially equal to that in compression.††

These conflicting opinions are no doubt due to the fact that so many circumstances influence the result. Some of these are the permanent set, previous loading, how the load is applied, whether in steps or continuously, repeated loading and unloading in order to obtain the final deformation for a certain load, the time element, internal stresses due to temperature, shrinkage, etc. In view of these facts it appears to the writer that the usual assumption, that Hooke's law holds, is to be preferred for the proposed exponential law for stresses which do not exceed the normal working stresses.

It is well known that Hooke's law does not hold for stresses approaching the breaking value and that this must materially influence the stress distribution in concrete arches the writer has pointed out elsewhere.‡‡ Bach has shown that for cast iron the modulus of rupture in bending is about 1.64 times the tensile strength and he explains that fact by showing that the modulus of

* *Bulletin No. 5*, Structural Materials Research Laboratory, Lewis Inst., Chicago, Ill., 1920.

† "The Materials of Construction," Fourth Edition (1912), Fig. 549-a, p. 604-a.

‡ *Loc. cit.*, p. 604-d.

§ Concrete Engineers' Handbook (1918), p. 250, quoting tests made at the Structural Materials Laboratory at St. Louis, Mo.; U. S. Bureau of Standards, *Technical Paper No. 2.*

‖ "Concrete, Plain and Reinforced," Second Edition (1912), p. 404.

¶ *Loc. cit.*, p. 408.

** "Taschenbuch für Bauingenieure," by Max Foerster (1920), pp. 241–242.

†† See, also, "Relationship Between Strength and Elasticity of Concrete in Tension and in Compression," by James W. Johnson, *Bulletin 90*, Eng. Experiment Station, Iowa State Coll., Ames, Iowa, May, 1928.

‡‡ *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 507, and Fig. 19, p. 508.

elasticity is not constant and is not the same for tension as for compression.* This difference brings about a shifting of the neutral axis toward the compression side, so that at failure, the compression at the extreme fiber is 73% greater than the maximum tensile stress. Recent tests on unreinforced concrete beams show that the modulus of rupture in bending is about 1.65 to 2 times as great as the tensile strength.† The exponential law using the same constants, k and m, for tension as for compression, as the author does, cannot explain this great discrepancy between the modulus of rupture in bending and the tensile strength, as the following calculation shows. The bending moment derived from the exponential law, Equation (9), is given by Bach‡ as,

$$M_e = \frac{2\,m}{2\,m+1}\,\sigma\,b\,\frac{h^2}{4} \qquad (112)$$

in which, σ is the maximum fiber stress, which occurs at the surfaces, the same for tension as for compression, since k and m are the same for both kinds of stresses; b is the width; h, the height; and m, the exponent for σ in Equation (9). For Hooke's law, $m = 1$ and,

$$M_h = \frac{b\,h^2}{6}\,\sigma \qquad (113)$$

The ratio of M_e to M_h is,

$$R = \frac{6}{4+\frac{2}{m}} \qquad (114)$$

If $m = 1.3$, $R = 1.08$, or the bending moment is only 8% greater than that found by Hooke's law. For $m = 2$, which is much too large, $R = 1.2$, which is still much too small. (Mr. Walker§ gives $m = 1.11$ as an average and Bach‖ gives values of m ranging from 1.145 to 1.207.) It is evident that the exponential law, with the same constants for tension as for compression, cannot explain the discrepancy between the modulus of rupture and tensile strength. When k and m are different for tension and for compression, as is the case with the cast iron investigated by Bach, to which reference has been made, the exponential law gives about 89% of the actual bending moment at failure, while Hooke's law gives only about 61%, assuming, in both cases, that the maximum bending stress equals the tensile strength.

The attempt to deal with the time effect, or the so-called plastic deformation, and its influence upon the stress distribution, is interesting. The author assumes that the exponential law holds for the elastic deformation and that another exponential law,¶

$$\rho = k\,\sigma^p\,t^q \qquad (115)$$

holds for the plastic deformations. From one set of tests by Fuller and More,

* "Elasticität und Festigkeit," Ninth Edition (1924), p. 298.

† Report of the Director of Research, Portland Cement Assoc., November, 1928, p. 170, and Table 9, p. 195.

‡ "Elasticität und Festigkeit," Ninth Edition (1924), p. 276.

§ *Bulletin No. 5*, Structural Materials Research Laboratory, Lewis Inst., Chicago, Ill., 1920, p. 88.

‖ "Elasticität und Festigkeit," p. 84.

¶ *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 70.

the author finds* $q = 0.42$, while from another set of tests by Mr. Earl B. Smith, he finds† $q = 0.18$; these latter tests appear to have been started when the concrete was only two weeks old, so that during the test the modulus of elasticity, strength, etc., must have been changing rapidly, making the results problematical.

The separation of elastic and plastic deformations seems an artificial procedure. Tests show the combined deformation only, and the stress-strain curve depends upon previous loading, so that the same specimen can not be subjected repeatedly to tests without introducing new factors which influence the result to an unknown extent. Moreover, the plastic deformations disappear in due course of time, and are not in any sense permanent deformations, except when the elastic limit has been exceeded. Bach reports‡ bending tests on wooden beams, which, when stressed to 2 100 lb. per sq. in. for 1.5 min., showed a remanent deflection of 0.10 mm. when the load was removed. After 22 hours of continuous loading, the remanent deflection was 0.34 mm. immediately after the load was removed, and 51 hours later this deflection had decreased to 0.14 mm. Tests on other wooden beams showed that this decrease in the remanent deflection was still going on after 162 hours of no load.

If the stress-strain curve is linear for plastic as well as for elastic deformations, Hooke's law applies, and the time factor affects the deformations but not the load stresses. The effect of the time factor is then to lower the modulus of elasticity and to decrease—very materially—the temperature stresses, since these are not suddenly applied. In order to investigate indeterminate structures properly, one would need to know the deformation as a function of the variable stress and time, and this function would probably be so complicated and its determination so uncertain, judging from the disagreement between authorities regarding the ordinary modulus of elasticity, that engineers would have to rely upon some approximate and simple function, such as Hooke's law, modified by the introduction of a time effect, such as the author suggests.

C. A. P. Turner,§ M. Am. Soc. C. E. (by letter).‖—Experience has led the writer to suppose that plastic flow after the removal of the forms was so small as to be incapable of determination by measurement with an ordinary 20-in. level. Mr. Straub apparently computes¶ the plastic deflection of a 140-ft. arch as 3 in.; and a 300-ft. arch of like relative dimension should be in the neighborhood of 6 in. for a time-flow period of more than 1 000 days.

For the purpose of making some independent observations, the writer ran a line of levels on a series of thirteen concrete arches that had been standing for about five years. Each has a rise of 1 to 4 and a span length, from center to center, of 304 ft. The third, eighth, ninth, and tenth spans differed little from the design grade. In other words, the floor was practically straight regardless of the fact that the levels were taken at a temperature 10 or 15°

* *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 71.
† *Loc. cit.*, p. 72.
‡ "Elasticität und Festigkeit," p. 111.
§ Cons. Engr., Minneapolis, Minn.
‖ Received by the Secretary, February 6, 1930.
¶ *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 112, Fig. 30.

below freezing. Other spans were not quite as close to grade on the curb line, but, since the curb was not exactly true, the total variation of ½ in. ± was not to be wondered at. The conclusions from these levels are as follows:

1.—That plastic flow capable of being measured definitely by a *Y*-level has not occurred in these thirteen spans.

2.—That, with a rise of 1:4, temperature changes affect the floor alignment only a negligible amount, although they may result in some slight difference in the elevation of the end abutments and the intermediate spans under wide ranges of temperature.

3.—That because the elevation and alignment of the floor were determined during construction by a 20-in. level, any alleged computation of changes of level which cannot be measured by the same instrument are either imaginary or without practical value.

4.—That the inadmissibility of the proposed theory from the practical standpoint arises from the misleading nature of the laboratory experiments upon which Mr. Straub founded his painstaking algebraic work.

Messrs. E. B. Smith,* F. R. McMillan,† and A. H. Fuller and C. C. More* were mistaken in supposing they had discovered a general phenomenon of plastic flow in concrete as applied to the practical structure. They had merely misinterpreted the law of static equilibrium of internal stress in its various manifestations.

Nearly all materials used for construction purposes are in a state of internal strain. These are of two kinds, residual and elastic. Static permanence of form subsists as long as internal elastic and residual strains are in equilibrium. Any cause upsetting this equilibrium is accompanied by a change in form and external dimensions of the specimen. Thus, a steel bar as received from the mill has been squeezed between the rolls and it leaves the mill in a state of internal strain. Cutting a series of saw kerfs across the top or bottom of the bar as it passed through the rolls will cause a slight curvature along the length of the bar. Similar treatment of bars bent cold in a horseshoe shape discloses the opposite nature of reactions of residual and elastic strain. This enables observers to account for the elevation of the primitive yield point of steel under over-strain and to dismiss fatigue as a phenomenon of residual strain unrelated to elastic stress and the selection of working stresses in rational design.

Contrary to algebraic theory, range of stress from tension to compression of a moderate amount, so far from injuring sound metal improves its elasticity, ductility, and tenacity as found by Bauschinger. This is brought about by the gradual elimination of residual strain by the work of the applied loads. The increase of the elastic constants is limited to 5% for *E*, nearly 20% for *K*, and no change in the modulus for shear.

Temperature change tends to reduce or eliminate residual strain and likewise also range of stress of moderate intensity. Conversely, range of stress

* *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 72.

† *Loc. cit.*, p. 70.

greater than five-eighths the primitive yield point increases or accumulates residual strain raising the yield point value.

The resistance of concrete follows the same statical principles involved in the discussion of steel. The 3 by 24-in. cylinders tested by Smith present equal cross-sectional areas subject to the rapid curing of the surface on the one hand and the slower hardening of the interior on the other. This condition differs widely from the 72 to 100-in. square sections of the arch, in which the surface effects are almost negligible.

Laboratory specimens, therefore, disclose a change of form accompanying the redistribution of internal strain which is practically wanting in larger cross-sections, as evidenced by the survey of the 300-ft. arches previously mentioned.

Briquettes cured in water for two or three years present tensile resistance equal to those cured in the dry if tested immediately; but if kept until half dried the resistance may be reduced 50 per cent. When fully dried, the resistance is restored, thus throwing some light on the time tests of Messrs. Fuller and More.*

The residual strain in the McMillan tests arise from a different cause.† In the initial crystallization of concrete, expansion takes place, inducing tension in the embedded steel. If the rods are as small as $\frac{5}{8}$ in. or less the tension as measured may be 7 000 to 10 000 lb. per sq. in. This tension in steel induces residual compression in the partly cured concrete envelope, having a reaction in residual strain opposite to elastic compression. Shrinkage accompanies the drying out of the concrete with elimination of steel tension, but the residual strain in the concrete remains until otherwise dissipated. Hence, the tensile element of resistance to flexure of the newly cured beam consists of the joint resistance of the residual strain in the concrete plus some tension in the steel. This residual strain is erroneously treated in textbook theory as direct tension in the concrete. A few repetitions of the load and it is largely dissipated. Temperature changes and time produce a like effect. Hence, the inadmissibility of applying such data to the practical problem of the arch rib.

In "Novum Organum", Bacon insists on clearing the metaphysics of technical science by observation, experiment, and experience. In his classification of the tendencies of the human mind to err he lists, under "Idola Tribus", the proneness of the human mind to suppose in Nature, an order and regularity greater than exists; to support preconceived opinion by affirmative observation, while omitting consideration of negative or opposite phenomena; to generalize from a few observations on the one hand or to give reality to mere abstractions or figments of the mind on the other hand. Under "Idola Theatri", he cites fallacious modes of thinking which from accepted systems of philosophy and erroneous demonstration, leaps to general conclusions.

No line of scientific endeavor needs clearing of the metaphysics of these two classes of error so badly as the common views regarding the elastic resistance of materials. The "Idola Tribus" of this viewpoint appears in the lack

* *Proceedings*, Am. Soc. C. E., January, 1930, Papers and Discussions, p. 71.
† *Loc. cit.*, p. 70.

of recognition of change of form caused by residual strain under the abstraction of the mind of the research fellow that it is nothing more than a measure of elastic strain.

Readings of deformation under load by an extensometer at the junction of a concrete slab and column are abnormally large. Considering such measurements as elastic strain instead of shrinkage recovery, an observer reached the conclusion that the stress just inside the column was approaching infinity. He stated that, because of this newly discovered fact, in a very short time when the elastic stress finally reached infinity, concrete buildings would fall apart and thus eliminate themselves from the field of competition.

Although shift of load in double bending by the combined action of torsional and bending couples is recognized in the old "Natural Philosophy" of Thomson and Tait, it is unheard of in the standard university textbook of to-day. The wide divergence between bending moment computed from steel stress and applied moment is attributed to the tensile strength* of concrete after it is broken in tension.

In line with Bacon's "Idola Theatri", colleges of engineering teach the student the formula of Hooke's law and feel they have accomplished enough in bringing the conception of the young engineer regarding elasticity up to within two centuries of advanced thought in physical science. The application of Hooke's law in flexure is then made by a system of partial differentiation arbitrarily assuming not only that shear strain does not follow Hooke's law, but that it is to be applied only to the horizontal components of the principal stresses, thus fixing the neutral axis at the gravity center of the section instead of at its natural locus along the length of the beam. Upon such half-way application of Hooke's law the student is taught computation of resistance to external shear force near the supports of continuous and simple beams, involving an inconsistency of 200 or 300 per cent.

Because longitudinal extension is accompanied by lateral contraction, the student is left in doubt as to whether this lateral deformation involves work. If it does not, is there not something wrong about Hooke's law? And if it does, is not the phenomenon in contravention of the law of conservation of energy? In the absence of instruction in the theory of the thermo-elastic properties of matter these are the confusing doubts which occur to the post-graduate student as observed in the technical press.

Confusion arises from a common but inexact statement that the internal work of deformation equals the external work of the load, when, in fact, this equality is limited to the specific direction of the applied force only. The total internal work is measured by the shift of the intrinsic internal heat energy of vibration from kinetic to potential, or *vice versa,* and is nearly double the external work. Hence, the textbook error in computing temperature stress.

* *Transactions,* Am. Soc. C. E., Vol. LXXXII (1918), p. 1560.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

RELATION BETWEEN RAIL AND WATERWAY TRANSPORTATION: A SYMPOSIUM

Discussion*

By Lewis C. Sorrell, Esq.

Lewis C. Sorrell,† Esq. (by letter).‡—The necessity of some intelligent attempt at forecasting long-run traffic developments, where large capital expenditures for transportation are contemplated, is matched by the extreme difficulty of doing so. The construction of a railway or improved waterway involves the investment of funds for many years; and the wisdom of doing this evidently depends upon the probable demand for such an instrumentality. Any economic unit, such as a nation or a region, is confronted with two questions, namely, how much transportation should be provided, and what instrumentalities should be created to furnish the service? Proceeding from a premise of competition, new facilities might be furnished wherever they could provide a service at less cost than existing agencies; and such generally has been the procedure in the past. A disposition exists to-day, however, to proceed from a premise of co-ordination; and this requires consideration of the effect of new facilities upon the pre-existing ones.

Co-ordination requires, in the case of transportation, a determination of the instrumentality best fitted economically to perform the task; or, otherwise stated, an allocation of the entire task of transportation among the several instrumentalities, so that the aggregate cost will be at a minimum. Considered generally, this viewpoint necessitates a calculation of the entire demand for transportation services, and of the cost of meeting that demand through furnishing different types of facilities. Considered at any given moment of

* This discussion (of the Symposium on Relation Between Rail and Waterway Transportation, presented at the Annual Convention, Milwaukee, Wis., July 10, 1929, and published in March, 1930, *Proceedings*) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Prof. of Transportation, The School of Commerce and Administration, Univ. of Chicago, Chicago, Ill.

‡ Received by the Secretary, February 1, 1930.

time, the question often becomes that of determining how increasing demand may be most economically satisfied. This is likely to be the case with a dynamic economic system, in which population and wants are increasing possibly at varying rates.

Most transportation facilities, such as highways, railways, and improved waterways, involve heavy outlays of fixed capital, which will be "sunk" for many years. Long-time investments necessitate long-run forecasting of the need therefor. Yet the difficulty of such forecasts cannot be ignored; and the longer the period of time, the greater the difficulty of an accurate forecast. Predicting traffic on American railroads for the next three months, or even for the next year, may be done with considerable accuracy; but attempts to state the probable traffic in 1940 or 1950 involve an appraisal of many elements, some of which are probably unknown to-day. The problem is essentially speculative; almost as much so as it was in the days when the transcontinental railroads were projected. Hence, one must approach such a task with two attitudes, namely, the methods used should be carefully scrutinized for possible errors, and the results even then cannot be accepted too tenaciously. One who essays such a task cannot be held to strict proof of the issues, for such an attitude would probably have prevented the consummation of many improvements in transportation.

Mr. Cornish* adopts the method of calculating the probable demand for transportation in the United States in 1950, and then considering whether a substantial portion of the increment might not be supplied at lower cost by improving the Mississippi River and its tributaries. Obviously, the two most significant terms of this method are demand and comparative cost. Demand he identifies largely with ton-miles, a unit customarily used to express the quantity of service rendered in carrying freight over the railroads. Two relatively minor limitations on this method may be noted. The unit of demand adopted for this study is customarily used, because it readily submits to statistical methods. Quantities carried may readily be combined with distance, in ton-miles and passenger miles; and records of these quantities are available for many years. This permits calculation of trends and the adoption of forecasting equations. It must not be overlooked, however, that the transportation service, which is the subject of demand, is much more than weight and distance; it is also a matter of total cost, frequency, dependability, in-transit service, reliability, financial responsibility, speed (or minimum over-all time), and legal liability. These qualitative aspects, as yet, have not been compounded into any unit. The caution must be injected, that a study of aggregate demand in terms of ton-miles will hardly afford a basis for determining what the apportionment of future demand is likely to be, as between railways and waterways, when that allocation must depend upon the free choice of shippers.

It has been customary, too, in attempting predictions of traffic growth to use railway traffic as evidence of the general development. In the past, when highways and waterways played a less prominent part than they do to-day,

* *Proceedings*, Am. Soc. C. E., March, 1930, Papers and Discussions, p. 563.

probably no large error resulted from the assumption that the general tendency was the same as that manifested in the growth of railway business; but the recent growth of highway and waterway traffic renders railway traffic less accurate as a barometer of total traffic. This consideration, together with the preceding one relating to the unit of demand, does not necessarily invalidate forecasts based upon them; it simply emphasizes the fact that tools heretofore used are somewhat inadequate for the purpose.

More serious is the assumption that underlies the method of measuring future traffic demand. The method used by Mr. Cornish conceives that the growth of demand for railroad transportation is a function of time. Because of the difficulty of forecasting demand, he concludes* that the "best solution lies in assuming that the trend of transportation progress in the United States during the past forty years will continue for the next twenty years." A time series of data is charted from 1890 to 1926; and the trend of that series is projected to 1950, at which date it is estimated that American railroads will probably enjoy a freight traffic demand of about 870 000 000 000 ton-miles.

It is instructive to compare these figures with several other attempts at long-range prediction. In 1924 John Balch Blood, Valuation Analyst of the Interstate Commerce Commission, prepared a study and forecast of railroad traffic—passenger and freight. He adopted the premise that such demand is a function of population, rather than of time; and from his study of the trend of population and traffic he concluded† "that ton-mileage varies approximately as the third power of population." His forecast provoked much criticism, especially as regards statistical methods, which were thought to yield extravagant results. The Bureau of Statistics of the Interstate Commerce Commission prepared a study, based upon the years 1900-1915 (selected to eliminate the extreme fluctuations of the World War and post-war periods), the results of which were utilized by the Commission in the reduced rate cases of 1922.‡ In 1923, the Chamber of Commerce of the United States made a report on the "Probable Freight and Passenger Traffic of the Railways of the United States of 1933". Of course these different attempts at forecasting used somewhat different methods and different terminal years. In 1924, L. A. Abbott, a Statistical Analyst of the Interstate Commerce Commission, discussed the methods used in the several estimates, and supplemented them with an independent estimate of his own, bringing them all down to the same terminal year 1943.§ The several forecasts yielded the following results:

Forecast for the year 1943 made by:	Billions of ton-miles.
Interstate Commerce Commission	494
U. S. Chamber of Commerce	727
L. A. Abbott	623
J. B. Blood	1 167

So far as concerns the demand for railway transportation, it is unsafe to assume that the trend for the next twenty-five years will be the same as

* *Proceedings*, Am. Soc. C. E., March, 1930, Papers and Discussions, p. 564.
† *Railway Age*, Vol. 76 (1924), pp. 369–371.
‡ Interstate Commerce Comm. Repts., Vol. 68, p. 701.
§ *Railway Age*, Vol. 77, pp. 243-247 (1924).

that for the last forty. A careful study of the growth of freight traffic on American railroads shows that a noticeable change in the rate of growth occurred about 1910. Dr. M. O. Lorenz, Director of Statistics of the Interstate Commerce Commission, recently made this statement:*

"From 1900 to 1914, freight ton-miles approximately doubled, while the ton-miles of 1928 were only about 50% greater than those of the fiscal year 1914. Furthermore, the passenger traffic and tons of freight have failed to maintain the same absolute amount of increase per annum in the second as compared with the first half of the period 1900-1928. Ton-miles of freight, however, show about the same annual addition in absolute amount from 1900-1914 (fiscal years) as from 1914 (fiscal) to the calendar year 1928, the annual addition having been 10.5 billions in the first period, and in the second 10.2 billions. It is not safe to predict the future solely by the past, because new forces come into action. The development of the motor vehicle was such a force, the effect of which on railroad traffic was not fully appreciated in 1915."

Generally speaking, it would seem as though methods of forecasting the trend of traffic, which are based upon the ratio of demand to population, would be somewhat more valid than those relying upon a time series alone. Mr. Abbott questions this,† holding that comparison of the different forecasts suggests that those using population growth as a basis have introduced an unnecessary element. It may be said that the time series does include all existing variables, without assigning weight to any single one. The writer has developed two equations of per capita ton-miles, using 1899-1926, and 1910-1926 as periods for the calculation of the trend. Linear equations were used in the following form:

(*a*) Fiscal Years, 1899-1926; Centered, 1912-1913:

$$Y = 2\,849.890 + 36.698X \ldots\ldots (1)$$

(*b*) Fiscal Years, 1910-1926; Centered 1918:

$$Y = 3\,227.0156 + 50.4473X \ldots\ldots (2)$$

in which, Y is the traffic, in ton-miles per capita; and X is the ratio of demand to population.

Equation (1) gives 4 207 ton-miles per capita, and Equation (2) 4 841 ton-miles per capita, as the expected demand in 1950. Mr. Raymond Pearl gives‡ the expected population of Continental United States at that date as 148 678 000. Applying this estimate to that of expected ton-miles per capita, gives 623 000 000 000 and 716 000 000 000, respectively. Most estimates, except those of Blood, would probably assign a considerably smaller value to the expected ton-miles for 1950 than the figure given by Mr. Cornish.

It is obvious that the actual demand for transportation is a function of many variables. Increasing population, together with an improving standard of living, would ordinarily tend to increase the demand for transportation, as expressed in ton-miles. There are numerous forces, however, that work in the opposite direction. Increasing decentralization of industry, substitution of transmission of energy for transportation of coal, reduction in the amount of

* As reported in the *United States Daily*, July 15, 1929.
† *Railway Age*, Vol. 77 (1924), pp. 243-247.
‡ "Studies in Human Biology," p. 590

coal or other raw material required in production, substitution of materials involving short hauls for those requiring long hauls, and importation of commodities formerly produced at home, are a few instances of factors that must be considered in estimating the probable future traffic on inland transportation facilities.

Cost of Transportation.—If the calculation of probable future demand is beset with difficulties, the computation of comparative cost is even more intricate. This is so, not only because cost data are so difficult to secure, but also because agreement may not exist concerning the items that should properly be included. That cost varies with the purpose to be served and the problem to be solved is well recognized. Where, as with transportation instrumentalities, a large volume of overhead costs exists, questions of policy concerning the apportionment of such overhead may complicate the solution. Likewise, joint expenditures, as for navigation and flood protection, may raise the issue concerning the proper sum to be charged against commerce. Where a long period of time is required to construct the enterprise, interest and operation deficits during such construction period must be considered. Similarly, the period of time required to develop traffic to the point, where the facility enjoys a degree of utilization necessary to realize the economies of its use, may accumulate a considerable burden of "developmental expenses", sometimes capitalized under the expression, "good-will".

The instant problem inolves a choice between two alternative methods of meeting an anticipated increase in the demand. Part of this assumed increase may be accommodated by an improved waterway, or by increasing the rail capacity. The choice of alternatives involves an approximation of the total economic outlay required for one instrumentality, as compared with that required for the other.

The method used by Mr. Cornish is reduced to five logical steps: First, he ascertains the probable increase in railroad investment necessary to afford the requisite increase in capacity. Second, he makes a similar estimate for the waterway. These two estimates are reduced to capital costs per ton-mile of capacity. Third, although not expressly so stated, he evidently regards rail operating costs as the present average per ton-mile earnings; this, of course, includes return on investment and taxes. Fourth, he takes the present earnings of the Inland Waterways Corporation as evidence of carrier-operating costs on the waterway. Fifth, he computes the saving to the shipper under the present 20% differential in water rates under the rail, on the assumed tonnage likely to be realized by the waterway, and adds thereto interest on the difference in capital outlay for waterway and railway, respectively. Limitations of space permit only cursory examination.

Railroad Capital Outlays.—Without attempting a detailed check on the trends in the capital outlays for the several kinds of railroad property, and without essaying the difficult task of estimating the amount of unused capacity now existing in the railroad plant, it may be said that an estimate of an annual investment of about $400 000 000 per year, to provide better freight-carrying facilities, would probably fall within any competent railroad

opinion. The exact amount of such outlays will depend not only on the needs of increasing traffic, but also on the desire to reduce costs by substituting capital for labor, and on the necessity of complying with public demand for terminal improvement and grade separations.

Waterway Capital Outlays.—Without checking the estimates of equipment and channel improvement costs, necessary to carry the estimated traffic increase, it may be observed that Mr. Cornish disregards capital outlays already made for river improvement and terminals, except the terminals used by the Federal Barge Line. Doubtless, it is difficult to estimate them, because, in part, they are joint with the cost of flood protection. Where a plant has been built, such that its productive capacity greatly exceeds the market, it may be a justifiable procedure to treat a considerable part of that capital outlay as a sunk cost; "a matter of historical interest but not affected by any subsequent acts of the management".* The premise of this argument is not a transportation agency larger than needed, but rather one to be enlarged to meet the demand. The outlays already made for the canalization of the Mississippi will be used to carry that increased traffic, as well as the future capital investment; and it would seem that the annual cost on account of total capital invested should be prorated against the total traffic anticipated in 1950.

Railway Operating Costs.—These, of course, are not identical with average freight receipts per ton-mile; but for purposes of rough comparison that is not very material. Mr. Cornish considers that the present average receipts of 11 mills per ton-mile are not fully compensatory to railroad capital; and assumes that increasing density will probably render the present freight rate level fully so. If railroads now existing in the United States can achieve a density of 1 600 000 ton-miles per mile of all tracks by 1950, assuming no other changes in cost, it is likely that a substantial reduction in rates could take place. This would lessen the savings due to waterway construction.

Waterway Operating Costs.—In this calculation Mr. Cornish takes the present ton-mile earnings of the Inland Waterways Corporation as evidence of carrier cost when using this agency. These figures do not include taxes and interest, which a private carrier must meet; possibly, he assumes that expected improvements will take care of these. In order, however, to present figures comparable with the rail carrier's earnings, the taxpayer's contribution should be considered. This would require something added for channel and terminal maintenance and operation, interest on the investment, and taxes foregone by the public on capital thus invested. With the present utilization of that waterway, the inclusion of such costs would probably produce a somewhat unfavorable result; but with the assumption of a much greater utilization in 1950, the added cost per ton-mile because of such overhead costs would be much smaller.

The Economic Benefits of the Waterway.—These are calculated by applying the present freight rate differential to the expected traffic in 1950, and adding thereto interest saved by reason of the much smaller investment required to carry 40 000 000 000 railway ton-miles by water than would be

* Economics of Overhead Costs," by Clark, p. 180.

needed to carry that traffic by rail. The estimate of an annual saving of about $160 000 000 would not be impossible of attainment, provided the assumed traffic density on the waterway is actually achieved; but the method of demonstrating this seems confused. The present differential of 20% is not based upon differences in operating costs; supposedly it is based upon the concession to the shipper necessary to induce him to use the waterway in preference to the railway. That is, it is more nearly a measure of the difference in the valuation of the two services; and, at present, that saving to the shipper is offset in part at least by the taxpayer's contribution in the shape of non-taxable and non-interest earning capital. The saving is not a net social gain; but even supposing that this differential, by 1950 did come to represent a net social gain, one would question the accuracy of adding interest saved to savings from freight rates in order to measure the total benefits. This is so because the freight rate differential then would be due largely to savings attributable to a smaller capital outlay for the waterway than for the railway.

A clearer procedure would be to ascertain the probable operating costs of the waterway carriers, not including returns on capital in the waterway itself, and to reduce these to a ton-mile basis. Next, calculate the capital investment in channel and terminals necessary to carry the assumed traffic. Compute the maintenance and channel operating costs, interest costs, and taxes that might be assessed against such a waterway itself; reduce these overhead costs to a ton-mile basis. Add this ton-mile toll to the ton-mile operating costs and thus arrive at the probable social cost of a ton-mile of water transportation in 1950 with an assumed utilization of 60 000 000 000 ton-miles per annum. For purposes of comparison equate waterway and railway ton-miles on the basis of 1½ to 1. The difference between this figure and the average probable railway freight receipts per ton-mile with the density premised for 1950, represents a net social saving due to the use of the waterway instead of the railway.

The remaining issues must be commented upon briefly. Although the expression "efficiency"* commonly means the ratio of output to input in terms of work, and may possibly be used to compare use with capacity to render service, nevertheless it seems preferable to adopt the term "utilization" for the latter concept, rather than the over-worked term "efficiency". In calculating the probable growth of waterway traffic on the Mississippi System,† it hardly seems wise to include the Panama Canal; as in the comparable case of railway location, traffic studies should be confined to agencies the circumstances of which are not too dissimilar. Should the estimated traffic fail to materialize, the burden of overhead costs per unit of waterway traffic would be considerably increased, and the savings, of course, correspondingly diminished. The same result will follow if the estimate of capital expenditures yet required is considerably below the actual sum needed.

This is in no sense captious criticism. Mr. Cornish has done good service in attempting to give a quantitative expression to the variables in this problem.

* *Proceedings*, Am. Soc. C. E., March, 1930, Papers and Discussions, p. 563.
† *Loc. cit.*, p. 572.

Transportation must be studied comparatively, with reference both to the service and the cost. Economic, engineering, and accounting methods must all be utilized in the process. The subject bristles with difficulties; and early attempts at solution will of course involve errors. It may not be too much to hope that further studies of this kind will follow; and that as a result something like a general acceptance of general principles may accrue, and that methods of solving these problems of transportation co-ordination may steadily improve.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

EDWARD AUSTIN BOND, M. Am. Soc. C. E.*

DIED DECEMBER 10, 1929.

Edward Austin Bond was born on April 22, 1849, at Dexter, Mich., the son of Hollis and Emily Faxon Bond. His ancestor, William Bond, came to the United States from England about 1620 and settled at Watertown, Mass.

Mr. Bond was educated in the public schools of Michigan and in a business college at Utica, N. Y. His first engineering work was for the Utica, Chenango and Susquehanna Railroad Company in 1868. In 1869, he served as Assistant Engineer on the Chicago, Pekin, and South Western Railroad, near Pekin, Ill., and on the Utica, Chenango and Susquehanna Valley Railroad, near Norwich, N. Y.

In 1870, he became Assistant Engineer on the Utica and Black River Railroad from Lowville to Carthage, N. Y., and also on the Black River and Morristown Railroad, near Philadelphia, N. Y. From late in 1870 to November, 1873, he was Chief Engineer of the Clayton and Theresa Railroad, from Clayton to Theresa Junction, N. Y. During 1874, Mr. Bond served as Resident Engineer for the Utica and Black River Railroad. Between August, 1874, and April, 1886, he was Chief Engineer of the Utica and Black River Railroad and during that time the line between Morristown and Ogdensburg, N. Y., was built. From May, 1886, to May 1, 1889, he was Chief Engineer and General Superintendent of the Carthage and Adirondack Railroad, on the portion between Carthage and Benson Mines, N. Y. Several railroads on which Mr. Bond was employed are now included in the New York Central Lines.

Mr. Bond was a member of the engineering firm of Hinds and Bond, of Watertown, N. Y., from 1889 to 1896, and during those years this firm designed or constructed water-works in Cape Vincent, Antwerp, Theresa, West Carthage, and Philadelphia, N. Y., and Barrie, Chatham, and Napanee, Ont., Canada. Mr. Bond had personal charge of the location of the Gouverneur and Edwards Railroad and of the surveys for the proposed Mohawk and St. Lawrence Railroad in Jefferson, Lewis, and St. Lawrence Counties, New York.

Mr. Bond was engaged in private practice from July, 1896, to November, 1898, as Consulting Engineer on various works. He was elected State Engineer for New York in November, 1898, and served as such until May 1, 1904. While Mr. Bond held this position many questions on canal and highway matters were considered. He served on the Committee on Canals that thoroughly investigated different projects in this field and reported to the Governor thereon in January, 1900. As State Engineer he had charge of a detailed survey of the canal routes through the State, known as the Barge Canal Survey,

* Memoir prepared by William B. Landreth, M. Am. Soc. C. E.

and reported thereon to the Governor early in 1901. The report gives in detail the various canal routes, estimates of cost, reports of the engineers in charge of the various parts of the work, and, in all, is a record of great engineering value. The studies covered by the report were so thorough and the methods used so exact that they received the confidence of the people of the State and enabled them to form a careful judgment on the matters considered. As State Engineer, Mr. Bond also had charge of the State's improved roads—then in their infancy—and many of his designs have since come into general use.

Mr. Bond resigned as State Engineer on May 1, 1904, to become Chairman of the Advisory Board of Consulting Engineers on the Barge Canal, and served as such until that Board was abolished by the State Legislature, in July, 1911. While the duties of the Advisory Board varied somewhat from time to time, it had a general oversight of the design and construction of the Barge Canal. Mr. Bond visited Europe as a member of the Barge Canal Terminal Commission that made an exhaustive study of terminals and in the main the report of that Commission has been followed on State Terminals.

After 1911, Mr. Bond was in private practice on canal and harbor works. He visited Uruguay on a harbor project in 1914, and was a member of the Advisory Board of the Trans-Alaska and Siberia Railroad.

He had the faculty of choosing competent and experienced men to work with him, and he showed much tact and wisdom in such choice. He quickly realized the essential points of a project and his decisions showed a full knowledge of the questions involved and their solution. He was a good judge of men and their work, and his associates showed their confidence in him by giving of their best efforts.

Mr. Bond was a Thirty-third Degree Mason, and a Republican in his political affiliations. He was an officer in several water-works companies in the United States and Canada.

He was married to Gertrude Hollenbeck in 1873; after her death to Clara E. Ellis, in 1904; and subsequent to her death to Mrs. Elizabeth Parsons, in 1908, who survives him.

Mr. Bond was elected a Member of the American Society of Civil Engineers on September 6, 1899.

FREDERICK AUSBERT DELAVAU, M. Am. Soc. C. E.*

DIED MAY 28, 1929.

Frederick Ausbert Delavau was born on August 25, 1871, in Rochester, N. Y., the son of Joseph Stansbury Delavau and Margaret (Clements) Delavau.

Mr. Delavau's entire life was spent in Rochester, where he likewise attended school. In 1889, he was employed by Gray and Storey, Civil Engineers, and subsequently, with William R. Storey, Civil Engineer, and with Mr. William G. Gray, on surveys and in private practice.

* Memoir prepared by C. A. Poole and John F. Skinner, Members, Am. Soc. C. E.

In 1898 he was employed by the City Engineer of Rochester as Draftsman and Transitman, from which positions he rose steadily through the various grades of Assistant Engineer and Office Engineer. In 1926, Mr. Delavau became Deputy City Engineer and, in 1928, Supervising Engineer in Charge of Local Improvements. He retired on a disability pension on April 16, 1929, after thirty-one years of service. After a year's illness he died at the Strong Memorial Hospital on May 28, 1929.

Mr. Delavau was a conscientious worker and led an exemplary life. In 1903, he was married to Sallie E. Williamson, of Skaneateles, N. Y., and is survived by his widow and one son, Robert W. Delavau, both residing in Rochester.

He was a member of the Masonic Order and the Shrine, the Rochester Engineering Society, and the American Society for Municipal Improvements.

Mr. Delavau was elected a Member of the American Society of Civil Engineers on January 17, 1927.

DANIEL WINGERD GROSS, M. Am. Soc. C. E.*

DIED JANUARY 23, 1930.

Daniel Wingerd Gross was born at Harrisburg, Pa., on September 25, 1871. He was the son of George A. Gross and Mary A. (Wingerd) Gross. Mr. Gross received his technical education at Pennsylvania State College and was graduated therefrom with the degree of Bachelor of Science in Civil Engineering in 1893.

He began work in 1893 as Rodman for the Beech Creek Railroad Company and his professional services before entering the employ of the Atlantic Coast Line Railroad Company were, as follows: In 1893, Rodman, the Beech Creek Railroad Company; 1894, Rodman, the Clearfield, Conemaugh and Western Railroad Company; 1895 to 1899, drafting and designing, Frog, Switch and Signal Department, the Pennsylvania Steel Company; 1899 to 1901, Division Engineer on construction and maintenance, the International Traction Company; 1901 to 1902, Chief Engineer, the Lake Shore Electric Railway Company; 1902 to 1904, Division Engineer on the construction of the Ballston Line, Schenectady Railway Company; 1904, Plant and Mine Engineer for Queens Run Fire Brick Company, Lock Haven, Pa.; 1905 to 1906, with J. G. White and Company, designing special work for electrification of the Sociedad Commercial Lines of Montevideo, Uruguay.

In March, 1906, Mr. Gross went to the Atlantic Coast Line Railroad Company as Engineer in charge of maintenance of way for its Florida Lines, with an office at Jacksonville, Fla. These lines had been acquired by purchase and merger a few years before and, consequently, there was need for definite constructive work by the engineer in charge of maintenance in order that they might conform to the standards of the Atlantic Coast Line System. So excellent were the plans of Mr. Gross for the improvement and so good was

* Memoir prepared by J. E. Willoughby, M. Am. Soc. C. E.

the work accomplished that he was soon promoted to the position of Engineer of Construction for the entire System, which position was created for him; his office was established at Wilmington, N. C. During the period in which he held this position the capital account expenditure for extension and improvement of the System for roadway, track, and structures aggregated more than $15 000 000.

When the Act of Congress was passed in 1913 to cover the valuation of the railways in the United States, Mr. Gross was put in charge of the work for the Atlantic Coast Line System and some of its subsidiaries, first, with the title of Chairman of Valuation Committee and, later, as Valuation Engineer. In this capacity, he directed the work of the valuation on the System until the period of Federal Control of railways, in August, 1918, when he was appointed Corporate Engineer. When the railways were released from Federal Control, Mr. Gross was promoted to the newly created position of Assistant to the President. In addition to the work then required of him as Executive he continued to give advice and attention to many of the valuation matters and was serving at the time of his death as a member of the Engineering Committee for the Southern Group of Railways in the Presidents' Conference Committee.

The work which Mr. Gross performed for the Atlantic Coast Line System during the twenty-four years of his service was of definite value to the property. By that work he established his reputation as a careful, intelligent engineer with such vision for the future as makes for economy. His personality was of a kind to win for himself a host of friends both in and out of railway service.

He died on January 23, 1930, and is survived by three sisters, Miss Helen Gross, of Wilmington, N. C., Mrs. J. M. T. Finney, of Baltimore, Md., and Mrs. John L. Yates, of Milwaukee, Wis. He never was married.

Mr. Gross was elected an Associate Member of the American Society of Civil Engineers on May 1, 1901, and a Member on June 24, 1916.

HENRY SCUDDER JAUDON, M. Am. Soc. C. E.*

DIED AUGUST 22, 1929.

Henry Scudder Jaudon, the son of W. A. and Mary Jaudon, was born in Savannah, Ga., on July 20, 1871. He was an alumnus of the Georgia School of Technology and Lehigh University, from which institutions he received his degree in Engineering in 1895 and 1896.

Prior to his graduation Mr. Jaudon secured employment, during vacation periods, with firms engaged in engineering projects of importance, and he profited by the contact thus formed. Following his graduation he was made Chief Draftsman and Inspector of the dry dock at Port Royal, S. C., Inspector for the Pottsville, Pa., Iron and Steel Company, and, later, was placed in

* Memoir prepared by the following Committee of the Georgia Section: J. W. Barnett, *Chairman*, and W. A. Hansell, Members, Am. Soc. C. E., and F. H. Frasuer, Assoc. M. Am. Soc. C. E.

charge of dredging operations for the United States Government. In 1898, he was employed by the American Water Works and Guarantee Company, of Pittsburgh, Pa., in the service of which he remained for five years, designing and supervising the construction of a number of water-works plants, including those at Birmingham, Ala., Shreveport, La., and Meridian, Miss.

In 1903, he organized the H. S. Jaudon Engineering Company, which operated extensively throughout the South. The efficiency of his organization was well known and brought him a lucrative practice. He pointed with pride to his achievements in more than a score of cities and counties where he had planned and directed municipal improvements and highway work.

As a citizen, Mr. Jaudon was public-spirited in an unusual degree. He took an active interest in matters pertaining to the betterment of his community and, as a member of the City Council and Chamber of Commerce of Elberton, Ga., contributed largely to the material advancement of the city of his adoption. He was an active member of the Methodist Episcopal Church and served on the Board of Stewards for a number of years.

His charming personality won for him scores of friends and admirers in all walks of life. His connection with professional organizations, and with social and fraternal orders, gave him a wide circle of acquaintances who mourn his passing. He was a member of the American Society of Municipal Improvements, the Florida Engineering Society, the Alpha Tau Omega Fraternity, the Rotary Club, the Masonic Order and the Benevolent and Protective Order of Elks.

Mr. Jaudon was deeply interested in his profession and those with whom he was associated. He never failed to reward the meritorious and lend a helping hand to the unfortunate. In his passing, the Society, his family, and his friends sustain an irreparable loss.

In 1900 he was married to Sarah Louise Arnold, of Elberton, in which city he resided until his death.

Mr. Jaudon was elected an Associate Member of the American Society of Civil Engineers on February 4, 1903, and a Member on February 1, 1910.

JOHN WILLIAM LIEB, M. Am. Soc. C. E.*

DIED NOVEMBER 1, 1929.

John William Lieb was born in Newark, N. J., on February 12, 1860, the son of John William and Christine Zens Lieb. He was educated in the local public schools and the Newark Academy, following which he entered the Stevens High School at Hoboken, N. J., graduating in 1876. He then entered the Stevens Institute of Technology, from which he was graduated with the Class of 1880, receiving the degree of Mechanical Engineer.

During his college course Mr. Lieb became interested in the new discoveries which at that time were being made public in the electrical field and shortly after his graduation he entered the employ of the Brush Electric Company in Cleveland, Ohio, where he worked on the arc-light systems as a

* Memoir prepared by Geo. A. Orrok, M. Am. Soc. C. E.

Draftsman. He soon saw that these systems had very nearly reached the limit of their development and after a few months, in January, 1881, he entered the employ of Thomas A. Edison, of the new Edison Electric Light Company, as Draftsman in the Engineering Department, at 65 Fifth Avenue, New York, N. Y. Here, he assisted in making the plans for the "Jumbo" dynamos and also in the design of the switches, regulating apparatus, and other electrical equipment for the new Pearl Street Station in New York which was to be the first central station in the world for the general distribution of electrical energy for utilization as light, heat, and power from an underground system of mains. Later in the year, he was transferred to Mr. Edison's Experimental and Testing Department at the Edison Machine Works, in Goerck Street, New York, and was engaged in researches and investigations on electric meters, regulators, and electrical apparatus, under the personal direction of Mr. Edison. Early in 1882, Mr. Lieb was assigned to the construction of the Pearl Street Station, in general charge of the planning and installation of the electrical equipment. When this Station was put into regular operation on September 4, 1882, under the auspices of the Edison Electric Illuminating Company of New York, he was given the appointment of Electrician in charge of its operation.

In November of the same year he was selected by Mr. Edison to proceed to Milan, Italy, to direct the design and installation of the first Edison Station in Milan; and on the organization of the Italian Edison Company (Societa Generale Italiana di Electricita, Sistema Edison), he was appointed its Chief Electrician, then Chief Engineer, and, finally, Technical Manager and Director of Stations. The Milan Station, which was put into regular service in March, 1883, was for a considerable time the largest and most successful electric light and power station in Europe. Mr. Lieb participated in the tests of the Goulard and Gibbs transformers at the Turin Exposition in 1884, and the Milan Company under his direction was among the first to install, in 1886-88, an alternating-current system of distribution, utilizing, among the very first applications, the connection of transformers in parallel according to the Zipernowsky, Deri, and Blathy patents, exploited by Ganz and Company, of Budapest, Hungary. He also did pioneer work in the transmission of high-tension alternating current at 2 400 volts, through concentric underground cables, to consumers located several miles from the central station. Under Mr. Lieb's direction there were also undertaken in Milan some of the first experiments in operating large direct-driven alternators in parallel.

The Thomson-Houston Series Arc System, with arc-light machines supplying 50 lamps in series, received here one of the earliest applications in Europe, and later was extended throughout the whole city from a separate arc-light station. The earlier arc-light poles had always been offensive to his aesthetic sense and Mr. Lieb inaugurated the use of artistically designed poles which were much more pleasing to the eye. In 1893, there was put into regular service in Milan under Mr. Lieb's direction, one of the earliest trolley car systems in Italy, furnishing regular service from the Piazza del Duomo to outlying districts. The Italian Edison Company, which was at first only an enterprise installing arc and incandescent lighting plants in cities and indus-

trial establishments all over Italy, later acquired its own manufacturing works for the production of dynamos, motors, and electrical apparatus of all kinds, including the manufacture of incandescent lamps. Mr. Lieb's interest in the incandescent lamp, which had manifested itself in his college days and continued during his work with Mr. Edison, was now used to good purpose, and his executive genius and organizing ability ensured the success of these establishments.

In the meanwhile he had returned to New York to be married to Minnie F. Engler, on July 29, 1886, taking her back with him to Milan where he continued his work in popularizing and extending the use of electric light, heat, and power. It was during this visit to America that Mr. Lieb became a member of the American Society of Mechanical Engineers. He was elected a member of the American Institute of Electrical Engineers the next year. In his connection with the Italian Edison Company, he served as adviser for many financial and industrial corporations engaged in the electric business where his active and enthusiastic mind united with the judgment of manager and executive, assured sound methods in the exploiting and extension of the electric industry. Already the possessor of three languages, German, French and English from early life, he now had become master of Italian, even speaking the local dialects with considerable ease.

In 1894, Mr. Lieb returned to America as Assistant to Mr. R. R. Bowker, then Vice-President and Executive of the Edison Electric Illuminating Company of New York. The next four years were most fruitful in the development of the use of electric light and power in New York City and Mr. Lieb made three visits to Europe with other officials of the company. This was preliminary to the design and construction of the Waterside Station and the adoption of high-tension alternating-current generation, with rotary converters and low-tension direct-current sub-stations for the Edison System. These years also saw the first storage battery installed on the Edison System.

On January 1, 1898, the control of the Company passed into the hands of Mr. A. N. Brady, with Mr. T. E. Murray as Vice-President. At this time, Mr. Lieb became Third Vice-President and General Manager, serving in this capacity until the Company's re-organization as the New York Edison Company in 1901. In the re-organized Company he became Associate General Manager, Vice-President, and General Manager and, finally, Senior Vice-President, which office he held at the time of his demise. During this period he had general charge of the installation and operation of the power plants and distributing systems in New York and was the executive in charge of the Company's technical operations, with supervision over all technical research and developmental work.

In addition to being Senior Vice-President and a Director of the New York Edison Company, Dr. Lieb was President and Chairman of the Board of The Electrical Testing Laboratories; Vice-President and member of the Board of Directors of the Yonkers Electric Light and Power Company; member of the Board of Directors of the United Electric Light and Power Company, and of the New York and Queens Electric Light and Power Company; Vice-President of the Edison Electric Light and Power Installation Company;

Director and member of the Executive Committee of the International Power Securities Corporation; Director of the Brush Electric Illuminating Company, the Empire City Subway Company, and the Consolidated Telegraph and Electrical Subway Company.

He was a Past-President and Fellow of the American Institute of Electrical Engineers; Past-President of the Association of Edison Illuminating Companies, National Electric Light Association, the New York Electrical Society, and the Edison Pioneers; Past Vice-President of the American Society of Mechanical Engineers and Fellow of the New York Academy of Sciences; Trustee of the Stevens Institute of Technology; Trustee and Vice-President of The Museums of the Peaceful Arts; and Trustee of the Italy-American Society. He was a member of the Illuminating Engineering Society; the Franklin Institute of Philadelphia; the American Association for the Advancement of Science; and numerous other professional and civic organizations, National and local. He was an Honorary Member of the Society of Italian Engineers and Architects, and of the Society of Italian Railway Engineers; Vice-President, Union Internationale des Producteurs et Distributeurs d'Energie Electrique; a member of the Elektro-technischer Verein, Associazione Electrotecnica Italiana, the Institution of Electrical Engineers of Great Britain, the Royal Society of Arts, and the Newcomen Society of London.

Dr. Lieb was for many years past the Chairman of the Lamp Committee of the Association of Edison Illuminating Companies, and during the World War was Chairman of the National Committee on Gas and Electric Service, representing in Washington, D. C., the public utility companies (gas, electric, and street railway service) of the United States in their co-operative relations with the various departments of the Government. He was a member of the Committee on Public Utilities of the U. S. Chamber of Commerce, on which he represented the National Electric Light Association as its National Councillor; Chairman of the Association's delegation to the International Chamber of Commerce; representative of the Association on the International Association of Producers and Distributors of Electrical Energy; member of the New York Chamber of Commerce and of its Committee on Internal Trade and Improvements; member of The Merchants Association of New York and Chairman of its Industrial Committee; member of the National Industrial Conference Board and Chairman of the Joint Fuel Committee, representing the National Public Utility Associations; member of the Committee to Study Plans for Co-operation between the War Department and the National Association of Manufacturers; and a member of the Committee on Standardization Survey, the American Society of Military Engineers, and the National Research Council.

He also served in international matters as follows: Chairman, Section "E", International Electrical Congress, St. Louis, Mo., 1904; Representative, U. S. Department of State at the International Railroad Congress, Rome, Italy, 1922; Member, U. S. Executive Committee, World Power Conference, London, England, 1924; U. S. Delegate, International Congress on Illumination, Geneva, Switzerland, 1924; Chairman, Reception Committee, International Electrotechnical Commission Meeting, New York, N. Y., 1926;

Vice-President, International Association of Producers and Distributors of Electrical Energy, Paris, France, 1928; and Executive Chairman, American Organization Committee, World Engineering Congress, Tokyo, Japan, 1929.

For his pioneer work in connection with the introduction of electric light and power service throughout Italy and the installation of the electric trolley system in Milan, Dr. Lieb was decorated by the Italian Government, being made Knight Commander of the Royal Order of the Crown of Italy and, later, being promoted to Grand Officer. In 1925 he was made "Ufficiale" of the Order of S.S. Maurizio e Lazzaro, and, in 1928, "in recognition of the services he has rendered in furthering professional industrial co-operation between French and American engineers and in facilitating the interchange between them of knowledge and experience regarding the construction and operation of central-station systems for the generation and distribution of electrical energy for light, heat, and power," was honored by the French Government, receiving the decoration of Officer of the Legion of Honor.

Dr. Lieb was most sympathetic with the youthful aspirant to a position in the Engineering Profession. He valued highly the training he had received at the Stevens Institute of Technology and maintained a very active interest in engineering education and the progress of his Alma Mater. He served as President of the Alumni Association in 1896-97 and as Alumni Representative on the Board of Trustees from 1908 to 1911. In 1921, he received, from his Alma Mater, the honorary degree of Doctor of Engineering, and, in 1927, became a life member of the Board of Trustees.

He was equally interested in technical Society affairs, becoming, as stated, a member of the American Society of Mechanical Engineers in 1886, and of the American Institute of Electrical Engineers in 1887. In the American Institute of Electrical Engineers he was elected Manager for the terms 1896-1899 and 1901-1903; Vice-President from 1899-1901 and from 1903-1904; and President in 1904-1905. In 1913, he became a Fellow of the Institute. He was elected Manager of the American Society of Mechanical Engineers for the term 1903-1906, and Vice-President for the term 1906-1908. He was greatly interested in the affairs of United Engineering Society and particularly of its Library, serving continuously on this Committee from 1909 to 1917. Outside these activities, he gave freely of his time and advice in the furtherance of the work of these technical societies.

Dr. Lieb delivered many lectures on engineering, industrial, and economic subjects in the leading universities and technical schools in the United States, and contributed many reports, papers, and discussions to the publications of professional and learned societies. On February 4, 1924, he was awarded the Edison Medal by the American Institute of Electrical Engineers for his work in connection with the development and operation of electric central stations for illumination and power.

As a manager and executive, Dr. Lieb held very definite opinions regarding the position of utilities or public service corporations as trustees for both the public and the investors. Coming into the business during its inception, he knew the experimental and speculative side and the long, arduous years before an adequate return became possible on the funds invested. As a public

service corporation is practically without competition, he held that service must be maintained as perfect as possible and at as low a rate as was consistent with fair returns on the investment. He early saw the futility and fallacy of municipal ownership blended with political control, and dependent on the tax fund to finance deficiencies. He saw his company paying a sufficient dividend to ensure the securing of additional capital when required and utilizing the best methods and devices for efficient and continuous service to the public at the lowest price conformable with a sound corporate policy.

As General Manager, the selection and administration of the Edison Company's personnel became his personal responsibility, which he considered as important as his responsibility to his stockholders and to the public. His wide sympathies were well known and his office was open to everybody, even to the youngest office boy, who had a grievance. Having worked as a mechanic, he knew the workman's point of view and respected it. His dealings with organized labor were sympathetic, but a suspicion of the "ca'canny" roused him to antagonism. He who gave of himself without limit and of the best that was in him, could not lightly suffer the cheat or "clock-watcher".

With the vision of the pioneer, the well considered knowledge and training of the scientist, and the wide sympathies of the humanitarian, Dr. Lieb endeared himself to every one with whom he came in contact. In a conference or argument, he had the faculty of picking out, as it were, the underlying idea, the main trend, the crux of a situation, and divesting it of all extraneous matter, going straight to the root of things. His reasoning was never oblique, but logical and cogent, sparing none, not even himself, in the strength of his analysis of a situation.

His sympathies were wide, ranging over the entire realm of human activities; his versatility and general knowledge might be termed encyclopædic. His English vocabulary was perhaps his most wonderful achievement, and his writings have always been models of clearness and good expression, in his use of the right word in the right place to obtain just the shade of meaning he desired.

While Dr. Lieb's first interest was always the electric industry, and particularly the Edison system, he had many hobbies, among which history, economics, art and archæology were perhaps the most important. His long residence in Italy, with its archæological monuments, and many survivals of mediæval times and the Renaissance, had given him a wide knowledge of these great periods of the world's history. He early became acquainted with the work of Leonardo da Vinci and studied his notebooks, learning the old Italian dialect in which they were written and becoming one of the foremost authorities on the work of that universal genius. He became greatly interested in the Etruscan monuments and the earlier Halstatt remains. His vacations were spent in research on art and history, visiting most of the sites of ancient cities and getting first-hand knowledge of the progress of civilization. He early joined the Raccolta Vinciana of Milan and maintained a most active interest in this Society. He was also greatly interested in the work of Perret, the volcanologist, and in the development at Llarderello, Italy, utilizing the interior heat of the earth for the production of power.

Dr. Lieb's reading was most extensive because he early acquired a capacity for rapid reading, which is decidedly uncommon at the present time. When this was added to a very retentive memory and a most wide-awake and far-reaching interest in both the modern and ancient world, the scope of his activities may be fully appreciated.

Mr. Edison, paying tribute to him, says:

"John W. Lieb was one of my most able assistants, and in the early days I used him in the development of the Pearl Street Central Station. He was a most energetic and active man and had such a comprehensive understanding of my electric lighting system that I selected him to go over to Italy and establish the Milan Central Station. He made a great success of the installation and of the introduction of the system into Italy. One of his notable achievements was the lighting of La Scala, the great opera house of Milan.

"His work in Italy was of a pioneering nature and led to the promotion of the great hydro-electric system which has been so extensively exploited in that country. His work there was but the beginning of a great and noteworthy career which, unfortunately, has come to an untimely end.

"He was a man of fine and sterling character, and I always had a very warm regard for him."

Dr. Lieb is survived by his widow; two daughters, Minnie E. and Julia C. Lieb; a son, Adolph W. Lieb, who was married in 1928 to Eleanor Wilcox; and a granddaughter. A brother, Oscar J. Lieb, and two sisters, Mrs. Charles Oscar Baldwin and Miss Anna Lieb, also survive him.

Dr. Lieb was elected a Member of the American Society of Civil Engineers on October 5, 1898.

WILLIAM CARY HATTAN, Assoc. M. Am. Soc. C. E.*

DIED MARCH 25, 1929.

William Cary Hattan, the son of Mark and Jennie E. (Siler) Hattan, was born at Kerrs Creek, Va., on December 15, 1875, of English (paternal) and German (maternal) ancestry. He was educated in the public schools and, later, attended Washington and Lee University, at Lexington, Va., from which he was graduated in 1899, receiving the degree of Bachelor of Science in Civil Engineering.

Mr. Hattan began his engineering work immediately after graduation, accepting his first position with the West Virginia Short Line Railroad Company as Rodman, Inspector of Materials, and Masonry Inspector. He was with this Company until March, 1901. From this date until August of the same year he was Masonry Inspector and Estimator for the Pennsylvania Railroad Company on the Wolf Creek Branch. From August, 1901, to January, 1902, he was in charge of a party doing general engineering work for J. H. Nichols, Civil and Mining Engineer, of Sharon, Pa. In January, 1902, Mr. Hattan accepted the position of Resident Engineer on construction with the Ozark and Cherokee Central Railway Company in Indian Territory. He was with this Company until January, 1903, at which time he returned to

* Memoir prepared by C. K. Lucas, Esq., Erwin, Tenn.

the East. From March to June, 1903, he was Assistant Resident Engineer on the Piney Extension for the Chesapeake and Ohio Railway Company, at Beckley, W. Va.

From June, 1903, until May, 1904, Mr. Hattan was associated with Vandevanter and Hood, Civil Engineers, of Baltimore, Md., in general engineering work, and as Assistant Engineer in charge of location and construction of the Baltimore and Belair Electric Railway. In May, 1904, he entered the services of the Western Maryland Railroad Company as Resident Engineer on the construction of the Cumberland Extension improvement work. While in the service of this Company he had charge of the erection of several large bridges, concrete arches, station buildings, and water tank and pumping station.

In the fall of 1905, Mr. Hattan accepted a position as Resident Engineer with the Carolina, Clinchfield and Ohio Railway Company and was given charge of a residency at the summit of the Blue Bridge Mountain where the construction work was very heavy. Although this residency was only 4.4 miles long, it included six tunnels which required three years to complete. His services on this work were very satisfactory, and at its conclusion he was transferred to the south end of the Carolina, Clinchfield and Ohio Railway where he had charge of the erection of one of the largest bridges built by this Company. He remained with this railroad until the construction work was completed and the road put into operation in September, 1909.

In April, 1910, he returned to the Western Maryland Railroad Company as Resident Engineer on construction. His ability was noted by the contractors on this work and in July, 1910, the Carter Construction Company appointed him Division Engineer in charge of sub-contract work on the road. He held this position until December, 1911.

When the Carolina, Clinchfield and Ohio Railway Company decided to build its Elkhorn Extension Mr. Hattan was made Division Engineer and placed in charge of the construction work on this 32-mile branch which, on account of the topography of the country and its inaccessibility, presented some very difficult engineering problems. On this extension there were eight steel and concrete bridges and twenty tunnels with an aggregate length of 20 786 ft., one of which, Sandy Ridge, was 7 854 ft. long, and lined throughout with concrete. This extension cost approximately $5 500 000, and upon its completion in June, 1915, Mr. Hattan accepted a position with the Kingsport Improvement Corporation. While with this Company he had charge of the municipal development and laid out the larger part of the present city of Kingsport, Tenn.

In June, 1916, Mr. Hattan entered the service of Thomas and Jones, Contractors, as General Superintendent in charge of construction work on the Missouri, Kansas and Texas Railroad Belt Line and its terminals at San Antonio, Tex. Upon the completion of this work in September, 1917, he took charge of the construction of Park Aviation Field, near Memphis, Tenn., where he was General Superintendent for the contractors, Thomas-Harmon Company. From February until November, 1918, he was Area Superintendent in charge of Area "C", U. S. Government Explosives Plant, at Nitro, W. Va., Thompson-Starrett Company, Contractors.

In December, 1918, he moved to Meyersdale, Pa., where he was engaged in private practice as Engineer and Contractor until July, 1920, when he accepted the position of Chief Engineer for the Carolina, Clinchfield and Ohio Railway Company (now the Clinchfield Railroad Company), with headquarters at Erwin, Tenn. This Company was very fortunate in securing the services of Mr. Hattan, as he proved to be a most efficient and accurate engineer with varied experience in both construction and maintenance work. He remained with this Company as Chief Engineer until his death which was caused by complications following an operation for appendicitis.

Mr. Hattan was a modest, unassuming, Christian gentleman who commanded the admiration, respect, and friendship of all who came in contact with him. He was an Elder of the Presbyterian Church; Assistant Teacher of the Brotherhood Bible Class; President of the Kiwanis Club of Erwin; and a member of the City School Board. He was also a member of the Tennessee Society, Sons of American Revolution, and of the American Railway Engineering Association.

He was married at Meyersdale, Pa., on April 12, 1912, to Sara Stein who, with a son and daughter, survives him.

Mr. Hattan was elected an Associate Member of the American Society of Civil Engineers on November 6, 1907.

EMIT COIN NICKEY, Assoc. M. Am. Soc. C. E.*

DIED MAY 5, 1929

Emit Coin Nickey, the son of L. F. and Addie M. Nickey, was born near Trafalgar, Ind., on March 14, 1882. He was educated at Ohio Northern University where he completed a partial course in 1904.

After leaving the University Mr. Nickey's first work was with the St. Louis-San Francisco Railroad Company as Rodman and Instrumentman until June, 1905. He then accepted a position with the County of Butler, Missouri, as County Surveyor, and was also engaged on townsite subdivision work in Southeast Missouri.

From June, 1905, until May, 1928, Mr. Nickey served as County Highway Engineer of Butler County, Missouri, having charge of all highway and bridge improvement work of that county and the financing of the same. In August, 1919, he successfully waged a campaign for a $500 000 county bond issue for roads and bridges, to meet Federal and State Aid funds for the 100-mile road system in Butler County completed at a total cost of approximately $1 000 000. In addition, he financed general county funds for road and bridge improvements each year approximated at $100 000 and prepared plans, estimate of cost, and made location surveys for all this work.

From August, 1913, until September, 1915, he served as Drainage Engineer in charge of surveys. He prepared plans and estimates of cost for Drainage Districts Nos. 10 and 11 in Butler County, and also for more

* Memoir prepared by E. C. Thomes, Esq., Poplar Bluff, Mo.

than 40 miles of ditches. The cost of this undertaking was approximately $130 000.

From May, 1905 to June, 1909, Mr. Nickey served as City Engineer at Poplar Bluff, Mo. Again, from January, 1927, until September, 1928, when he resigned, due to ill health, he was City Engineer of Poplar Bluff, in charge of all the engineering work within the city, including a street paving program estimated at $400 000. He also was engaged on sewerage and other drainage work. From 1905 until his death in 1929 he served as County Surveyor of Butler County.

Mr. Nickey was a man of high ideals, of sterling character and of the highest ability. Of pleasing personality, he possessed a genial disposition, endearing himself to all who knew him. He was a member of the American Association of Engineers, and a member of Lodge No. 209, A. F. and A. M., of Poplar Bluff. He was also a member of the Benevolent and Protective Order of Elks, Poplar Bluff Lodge No. 589.

Mr. Nickey moved from Missouri in October, 1928, to Colorado where he died on May 5, 1929, of cerebral hemorrhage. The funeral rites were held under the auspices of the Colorado Springs Lodge No. 76, A. F. and A. M., and Colorado Springs Commandery, Knights Templars; interment was in Evergreen Cemetary, Colorado Springs, Colo.

He was married in Poplar Bluff, on February 14, 1905, to Bessie Flanigan, who survives him.

Mr. Nickey was elected an Associate Member of the American Society of Civil Engineers on November 25, 1919.